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Concrete Dome for the New Technology Buildings

The crowning architectural feature of the new buildings for the Massachusetts Institute of Technology on the banks of the Charles River, Cambridge, Mass., is to be a large dome surmounting the central one of the group, which is located at the extreme inside end of the forecourt leading up from the river. While externally this dome, with its building, will be of limestone in architectural keeping with the rest of the buildings, its structure is of reinforced concrete, the same as the other structures. The view on this page shows a recent stage in the building of the dome, which is now almost completed.

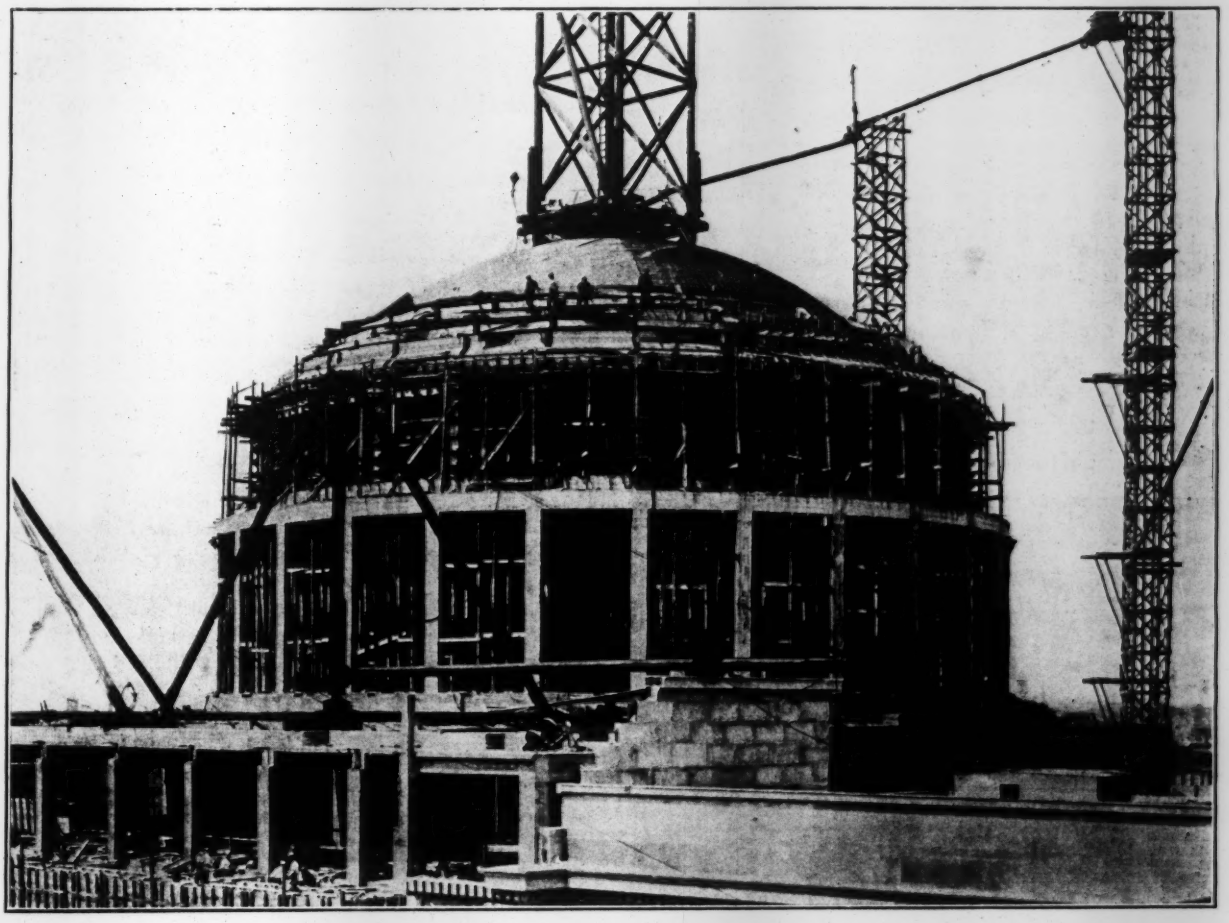


FIG. 1. CONCRETING THE DOME OF NEW TECHNOLOGY BUILDINGS, CAMBRIDGE, MASS.

That portion of it which rises above the five-story reinforced-concrete building upon which it is located consists of two tiers of cylinders, or drums, surmounted by the dome proper, the lower half of which is in the form of steps and the upper half in the form of a truncated sphere the flat top of which has an opening to admit light to the interior. The drums are supported on two circumferential rows of columns, the outside row of the lowest drum resting on a ring girder spanning the beams of the fifth floor of the main building, and the outside row of the upper drum spanning radial beams connecting the outer and inner columns of the lower drum. The inner columns of the two cylinders are continuous through to the building below, being located on the corners of an octagon.

The step portion in the lower half of the dome is of reinforced-concrete slabs formed to step shape and resting on radial inclined girders spanning the outer and

inner columns of the upper drum. The upper part of the dome is a portion of a sphere, and abuts at its lower plane on a ring girder spanning the inner columns of the drums, the upper plane being formed by a ring girder on the rim of the top opening. This slab varies in thickness from 5 in. at the top to 8 in. at the bottom, and is reinforced with a cross-webbing of twisted rods of the type shown in the view, Fig. 3.

The dome is 147½ ft. above the general level and 65 ft. above the parapets of surrounding buildings. The diameter of the lower cylinder is 120 ft. and its height 37 ft. The diameter of the upper cylinder is 108 ft. and its height 18 ft. The spherical portion of the dome, including the steps, is 23½ ft. above the top of the upper drum and is curved to a radius of 56½ ft. The two hemispherical dome portions are 70 ft. in diameter.

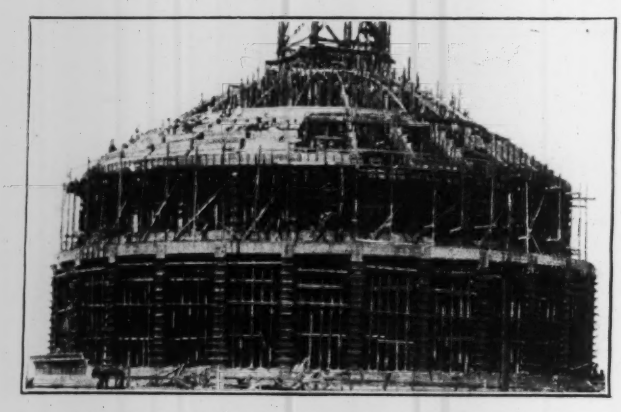


FIG. 2. FORMWORK ON TECHNOLOGY DOME

The view in Fig. 2 shows an earlier stage in the formwork and illustrates fairly well how the dome was carried up. The main framework presented no particular difficulties, inasmuch as it could all be supported from the ring girders, but the central spherical dome had to be supported from the roof of the main building, which required thoroughly cross-braced falsework posts from 50 to 70 ft. high. The lower side of this base form for the roof of the dome was made in a continuous operation, forming, in fact, a complete timber dome, as shown in Fig. 2. On this the reinforcement was laid above the step portion while the step portion was poured against vertical forms, the tread of the steps being screeded. This lower section is shown to the right in the dome in Fig. 1. The upper forms for the spherical portion of the dome were laid in the form of small panels of varying shapes to accommodate them to the radius of the dome, 21 of these being used at a time. When the group of forms had been placed, the segment was poured all at once and the operation repeated on the next segment; six segments were required to complete the dome.

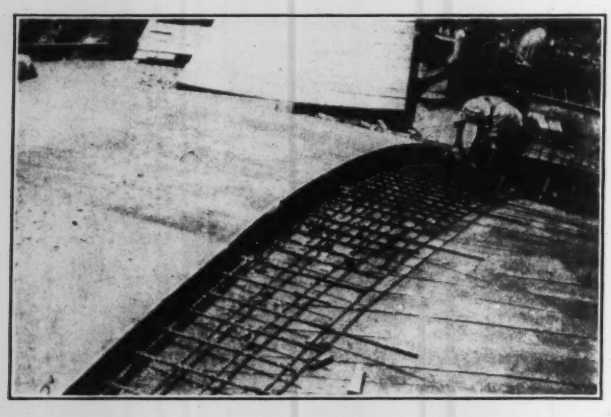


FIG. 3. NEAR VIEW OF REINFORCEMENT IN PLACE ON DOME

Concreting was done from a tower 190 ft. high, located about 100 ft. away from the dome center, the chute being supported, as shown in Fig. 1, by an intermediate tower located on the dome proper. The central tower shown in the views is a continuous timber tower 18 ft. square and 119 ft. high, built up from the roof of the main building. To each corner of this tower is attached a boom, which is used for the conveying of forms and the limestone covering which is now being placed on the dome.

The buildings of the Massachusetts Institute of Technology are being built by the Stone & Webster Engineering Corporation, 147 Milk St., Boston, which firm is in charge of the engineering design as well as the construction. William Welles Bosworth, of New York, is the architect.

The Largest Tunnel in the World

The largest tunnel ever undertaken is that which has been partly driven at Marseilles, France, for the Marseilles-Rhône Canal. The tunnel is 72 ft. wide and the height from the bottom of the tunnel to the crown of the arch is about 40 ft. The total length when completed will be about four miles, and when its great cross-section is taken into consideration in connection with its length, the total volume of excavation will doubtless be greater than that of any tunnel for traffic purposes ever constructed. The total amount to be excavated is in the neighborhood of 2,500,000 cu.yd. E. L. Corthell, in the course of a visit to notable engineering works in the Mediterranean countries, examined this tunnel two years ago and gives a brief description of it in *Professional Memoirs of the Corps of Engineers* for May-June, 1915.

The accompanying drawings show cross-sections of the tunnel and the general method of construction, with a map of the navigation scheme of which the tunnel forms a part. A part of Mr. Corthell's description of the work is as follows:

We arrived at Marseilles on Feb. 22, 1915. Mr. Batard, Chief Engineer of the Port, and Mr. Bourgougnon, Chief Engineer of the Marseilles-Rhône Canal now under construction, were on the dock to meet us and take us first in Mr. Batard's sea-going launch about 5 mi. (8 km.) along the route of the new canal, the terminus of which is in the port of Marseilles, straight to the entrance of the tunnel now under construction through the rocky ridge that separates the Bay of Marseilles from the "Etang de Bolmon," and thence along the shores of the large "Etang de Berre," about 8 mi. (12.8 km.) in all, and thence along the outlet of these lakes to the "Golfe de Foz," along the northern shore of that water direct to ancient Arles, following the latter part of the way the course of the Rhône.

The total length, Port of Marseilles to the Rhône at Arles, is somewhat less than 50 mi. (80.5 km.). It is built to bring the Rhône River traffic from Lyons and above direct to Marseilles. It is being built by the French government, with some assistance from the Chamber of Commerce of Marseilles. The large Rhône barges can then avoid the dangerous route of 25 mi. (40.2 km.) around Cape Couronne from the mouth of the Rhône, or rather from the mouth of the St. Louis Canal to Marseilles.

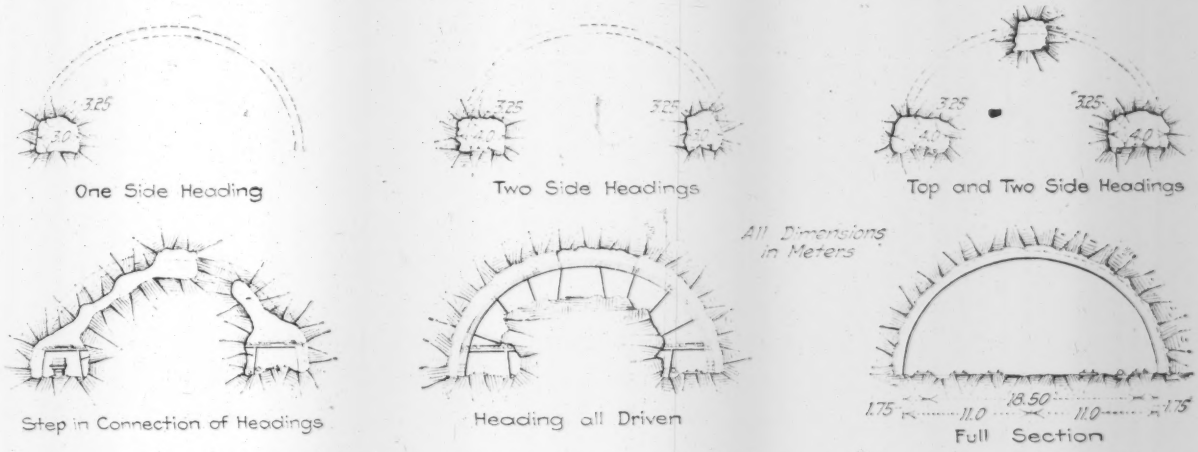
Before describing the tunnel, which is the principal feature of interest, let me say that the mouth of the Rhône has an interesting history. For centuries the question of improving it for the Rhône navigation had been a matter of study, projects and efforts, until the government started to build dikes to close the smaller outlets of the delta and two jetties at the principal mouth. But two serious errors were made, the jetties were too far apart and the methods of construction were too slow. The silt and sands protruded beyond the advancing jetties, always in advance, the bar moving outwards until, weary of "chasing a rainbow," it was decided to build a lateral canal (Canal St. Louis), about 3½ mi. (5.6 km.) long to debouch into the Gulf of Foz.

The disappointment and its causes are shown, with sketches, in my "History of the Mississippi Jetties," published by John Wiley & Sons in 1880. The conditions and history were carefully examined by Captain Eads at the time of his visit to Europe in 1874 to learn the facts about many improved river mouths, to support his arguments for the bill then before Congress for improving the mouth of the Mississippi River by means of jetties. His opponents were using the history of the mouth of the Rhône as an argument in favor of the Fort St. Philip Canal from the Mississippi to Lake Borgne and against the jetty proposition. Now, the Canal St. Louis is to be practically abandoned and one 50 mi. (80.4 km.) long is to be built.

The tunnel is probably one of the most interesting rock tunnels ever undertaken in that it is, no doubt, the largest and one of the longest barge-canal tunnels in rock. The tunnel, lined throughout its nearly 4½ mi. (7.2 km.), is 22 m. (72.2 feet) wide inside the lining, and the crown of the arch (nearly semicircular) is 9.90 m. (32½ feet) high. The depth of water is to be 3 m. (10 feet). The plans illustrate the whole process of excavation, centering and lining, and are well worth the study of tunnel and canal engineers.

Briefly, as the detail plans are too extensive and numerous to be published entire, the process is as follows: A heading on the left at the base 3x3.25 m. (9.8x10.7 ft.), then one on the right at the same level and same size, then the enlargement of the two inward to give room for a track and ditch for drainage, then a small heading at the summit of the arch, then the enlargement upward of the two side headings to connect with the top heading, with a staging to receive the falling rock to be dropped into small dump-cars on the track, the permanent lining at the same time started on each side, then the enlargement of the excavation under the roof on both sides, which gives a chance to put in heavy centering on short posts, resting on the same mass core of unexcavated rock, then the masonry lining over the advancing centering completed, the centering then removed and carried forward and the bench removed.

The work is done by compressed air and it has the usual ventilating machinery and pipes, compressed-air pipes, drainage ditches, etc. The rock is quite varied in character: Dolomite of the Upper Jurassic, limestone, sequannian (above the Jurassic), and a mixture of the two. Suitable stone for the lining is found in the materials removed by the tunnel excavation. The contractor for the entire work is M. Chagnaud, who has recently completed the Loetschberg tunnel and approaches between Bern and the Simplon.



CROSS-SECTIONS OF ROCK TUNNEL ON MARSEILLES-RHONE CANAL, SHOWING SUCCESSIVE STEPS IN EXCAVATION AND LINING

At the time the sections were taken which are here reproduced (July 1, 1912), the tunnel had been completed and lined for a distance of 600 meters and the advance headings were driven 500 meters farther. It is perhaps needless to say that work upon the tunnel, as on practically all other constructive engineering works in the belligerent countries of Europe, is now suspended.

It is doubtful whether it will ever be resumed. The vast burden of debt due to the war will prevent the European nations from embarking on the construction of any public works not absolutely necessary for a long time to come.

Apart from this, however, the proposition to build this enormous and enormously costly tunnel, merely to save the dwindling traffic by river barges a few miles of navigation in the open Mediterranean, would appear to be a scheme absolutely quixotic from the economic point of view. Probably the tunnel will take its place with other abandoned engineering dreams.

MAP OF THE PROPOSED MARSEILLES-RHONE CANAL

Coast Erosion and Protection on Long Island and New Jersey--II

By Gerald O. Case*

SYNOPSIS—Takes up in detail the causes of coast erosion and illustrates the effects with general cases and specific references to the New Jersey coast. Theory and behavior of sand dunes.

Causes of Coast Erosion

A study of the causes of coast erosion is essential before it is possible to say what are the best means of combating it. Erosion is due to many causes, and coast protection works cannot be expected to be successful unless they are designed after a careful consideration of the causes of erosion at the particular place it is desired to protect. Many of the coast-protection works constructed on the New Jersey coast have proved a failure, and this has undoubtedly been due in a great measure to an imperfect knowledge of the causes of the erosion. The author has known of cases where the erosion was due to wind action in moving inland the natural sand embankments but where the protection works have been designed and constructed to prevent erosion by wave and current action. Of course, they were a failure and the erosion continued, which could have been certainly stopped by very inexpensive works had the correct cause of the erosion been recognized.

Subsidence of the Land

A new cycle of coastline changes are inaugurated by any important alteration in the relative levels of land and sea. Subsidence of the land or rise of the sea will, of course, cause erosion. Although there are evidences of local changes in the tide level on the New Jersey coast, there is no evidence of a gradual sinking of the land, as has been supposed by many people. Recent precise leveling in the vicinity of New York harbor confirms the surveys made in southern New Jersey, by proving an absence of any tilting of the land, and shows further that mean sea level is the same now as it was more than a quarter of a century ago at Sandy Hook.[7] The forces—wind waves, wind currents, tidal currents and wind action—now operating on the New Jersey and Long Island coasts are fully capable of causing the great changes which have undoubtedly taken place in recent years.

Inequalities in Amount of Littoral Drift

There is practically no such thing as static equilibrium of a foreshore, the surface layers being constantly in motion. When a shore is met with which is in apparent equilibrium, neither erosion nor accumulation taking place, it does not follow that movement is absent, but simply that removal and deposition approximately balance each other. Anything disturbing this balance at once becomes evident in a gain or loss to the foreshore. The effect of obstructions, such as a headland or a break in the continuity of the coastline, which will interfere with the littoral drift is therefore very important. Wherever an obstruction checks littoral drift, erosion very often takes place, under natural conditions, on the leeward shore.[8]

There are also peculiar conformations of the coastline which allow a certain part to be sheltered from particular winds, which are active in eroding material from other parts of the coast. In such a case littoral drift may commence at the particular point where the shelter, say, of a headland, ceases to be effective, thus producing severe erosion owing to the absence of drift from the other side to fill the place of the material removed.

The amount of littoral drift taking place on any part of the coast is dependent partly on the gradient of the foreshore and sea bed. With a very flat gradient the tendency is for the ends of oblique waves, on entering the shallow water, to be retarded and for the line of waves to swing round and come in at right angles to the coastline. The flatter the gradient, the less material will be moved by wave action per unit area. Erosion of the coast, under natural conditions, tends to flatten the gradient, and accretion to steepen it. If from any cause a shoaling of the sea bed takes place on any part of the coast, then the tendency will be for accretion to take place on the foreshore opposite the shoaling, and for erosion to take place on the leeward shore, due to interference with the alongshore supply of drift.

The general tendency is for the sea to erode headlands and projections. Reference to Fig. 3 will show that waves *G* which roll in parallel to the general coast line are oblique to the flanks of the headland and so tend to transport material from the point to the shore on either side—that is, from *C* to *B* and *C* to *D*. The action of oblique waves from either side also tends to move material away from the point of the headland. If the predominant waves causing littoral drift on the coast are coming from the direction *F*, most of the movement will be from *C* to *D*, but oblique waves *H* which may sometimes be formed will also cause a movement away from the point, in this case from *C* to *B*.

Given a straight coastline composed of alternate layers of hard and soft material, then the effect of the sea will be to form bays and headlands by erosion of the soft material; but after a time, perhaps several years, the projecting headlands would shelter the bays, and erosion of the soft and hard strata might then take place at the same rate. There is also a limitation to localized erosion due to other causes, and it is very important to bear this in mind. If from any cause, such as interference with littoral drift, erosion is caused along a certain length of coast, then if this continues a bay will be formed, the direction at which the waves strike the coast and also the gradient of the shore will be altered and the headland will shelter the bay. There are

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[7]Annual reports of the New Jersey State Geologist, 1912, 1913.

[8]The leeward side of a natural or artificial obstruction to littoral drift is the side sheltered from the wind waves causing the predominant drift, and the windward side, that exposed to the wind waves causing the drift.

therefore three things tending to stop the localized erosion and cause erosion to take place at the same rate all along the coast.

An outlying island has an important effect in interfering with littoral drift, causing accretion to take place between the island and the mainland and erosion to take place on the leeward shore. This action is seen by reference to Fig. 4. It is obvious that the coast from *B* to *C* is sheltered from the predominant oblique wave action; there is, therefore, a constant feeding in of material from *AB* and accumulation takes place between the island and the mainland. Material eroded from the sea face of the island also tends to accumulate in the form of spits built out toward the mainland. As little or no material will be moved on to the foreshore between *C* and *D* to make up for what is moved away from there by oblique wave action, erosion will take place. This effect may also be seen if a wreck is stranded near the shore. When the island has become joined to the mainland and a headland thus formed, the erosion of the sea face of the island will still continue, the material

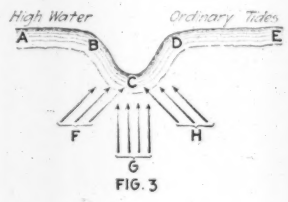

FIG. 3

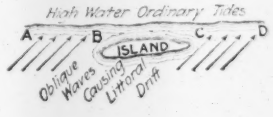

FIG. 4

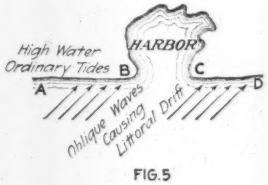

FIG. 5

FIG. 6

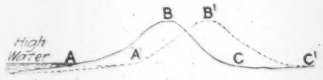

FIG. 7

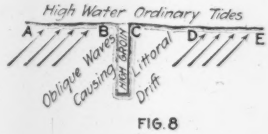

FIG. 8

FIGS. 3 TO 8. ACTION OF WAVES CAUSING DRIFT

from which will then be moved to the shore between *C* and *D*, causing the localized erosion there to stop and the coastline to be gradually straightened out again.

Thus there are some cases of localized erosion which may go on for some years, but which will inevitably be stopped in course of time by the operation of the laws of nature.

Erosion on the Leeward Side of Inlets

Where there is a predominant littoral drift, a large supply of material, and a river outlet cutting through the coastline, erosion as a rule takes place on the leeward side. When the drift encounters the mouth of the river or inlet it is trapped and carried into the inlet or out to sea. The flood tide, setting into the inlet with much greater velocity than on the open coast, carries in sand to form shoals and silt to form salt marshes, while the ebb current, on the contrary, tends to transport material out to the sea bed. Referring to Fig. 5 it will be seen that owing to the littoral drift being intercepted at *BC* little or no material may be moved to the leeward shore *CD* to replace that moved from there by oblique wave action.

If the amount of littoral drift traveling along *AB* toward the inlet is greater than the flood tide can move into the inlet or the ebb transport to seaward, as is often the case, then a spit is formed part of the way across the inlet, from *B* toward *C*, or in some cases is built right across and closes it up, when material can once more be transported to the shore between *C* and *D*, and the erosion there will then cease. All the time that the inlet exists part of the supply may be often cut off from the leeward shore *CD* and erosion often takes place.

The spit on the north side of Barnegat inlet has grown out in a southerly direction, and a very considerable amount of erosion at the same time took place at the south side of the inlet. A comparison of the surveys of 1840 and 1886[9] shows that between these dates the leeward shore at the south side was cut back for a distance of about 2300 ft., and the east shore eroded from 100 to 1000 ft. for a distance of 1¼ mi.

The coastline changes which have taken place at Hereford and Turtle Gut inlets also illustrate the general rule that when accretion takes place on the windward side of inlets, erosion takes place on the leeward side. Between 1842 and 1901 the shore at Anglesea (Five Mile Beach) on the leeward (south) side of Hereford inlet was considerably eroded—in places the high water line advancing 1500 ft., while considerable accretion took place on the windward (north) side. The same thing happened at Turtle Gut inlet between 1842 and 1901. Five Mile Beach on the windward side was built out in a southerly direction, and at Two Mile Beach on the leeward side extensive erosion took place at the same time.[9]

In places where the windward side of an inlet is being rapidly eroded owing to inequalities in the amount of littoral drift or other causes, then the tendency is for accretion to take place on the leeward side. This is partly due to the action of eddy currents depositing material in the form of shoals and partly to alterations in the conformation of the coast. Thus at Absecon, Great Egg and Corson Inlets rapid erosion has, in recent years, taken place at the north sides and some accretion at south sides of these inlets.

When a spit is formed in front of a harbor or inlet into which a river discharges, and, as frequently happens, it closes the inlet and in course of time a new entrance is opened by storms and floods on the windward side, we may then have a spit on both sides with the entrance at or near the middle (Fig. 6). By the action of littoral drift the windward spit *BC* will at once commence to grow and pushes the entrance gradually toward *D*. The extremity of the spit on the leeward side *D* will then be gradually eroded. This erosion may be partly due to the river cutting off the supply of littoral drift and partly to the action of the ebb currents concentrated on its end. If, as is often the case, the amount of accretion on the end of the windward spit is more than the

[9] Annual report of New Jersey State Geologist, 1885 and 1905.

amount of the erosion on the end of the leeward spit, then the spits will gradually unite again and close the inlet.

The northerly movement of Shark River spit has frequently closed up the inlet there, and when a new entrance has been formed further south by floods and gales, the entrance has immediately begun to move northward again; the spit on the southern side increasing in length by littoral drift, and the southern end of the spit on the north side being at the same time eroded. At Manasquan River the same thing has taken place.

Effect of Onshore Winds

Winds blowing directly onshore cause a denudation of the upper part of the beach to take place, and a consequent advance of the high-water line results. This is due to a surface current being formed in a landward direction, which is compensated for by an undertow seaward. At the same time the waves break further seaward and are also forced shoreward by the wind, having their frequency of impact increased. There is no time for the regular sequence of plunge, uprush and backwash to take place; the waves hurry in on top of one another, and a surf is formed. The character of the wave also alters from an oscillatory to a translatory at a considerable distance from the shore, thus aggravating the surface drift due to direct wind action, and consequently the undertow seaward. In this state the sand is churned up for some distance from the shore, and naturally the quantity in suspension is greatest near the bottom, where the undertow is at work transporting it seaward.

Direct Impact of Waves

The direct impact of waves is a potent factor in the disintegration and removal of cliff material, and its action can be observed during gales on any part of the New Jersey coast where there is a cliff or bluff exposed to wave attack. During gales at Avon the waves have sometimes struck the 10-ft. high cliff with sufficient force to cut it away for 50 ft. and send the spray 40 ft. high into the air.[10]

Large masses may also be torn off by the alternate compression and expansion of the air in crevices exposed to the wave stroke, or by the transmission of the blow of the waves to the sides of fissures and cavities which are filled with water. The destruction of cliffs is chiefly brought about by the undermining action of the waves aided by subaërial agencies—rain, frost, etc. The constant percolation of rain water through cracks tends to disintegrate the cliffs, and the action of frost is well known in pulverizing the surface and splitting off fragments. The continued erosion is, however, dependent on the waves and currents moving alongshore or seaward the material eroded by wave impact, and thus allowing the waves again directly to attack the cliff. If the undermining action is prevented by the accumulation of débris at the base, then the cliff will in time assume an inclination of repose, due to the action of subaërial agencies.

Formation and Movement of Sand Dunes

On the sandy coasts of New Jersey and Long Island, wherever there is a fairly wide expanse of sand dry at low tide, there is a tendency for sand dunes to form above high-water line of ordinary tides. At low tide a certain amount of sand is dried by the wind and sun, and as soon as it becomes dry the wind is able to transport it inland. Onshore winds are more effective in moving sand inland than the weaker and less frequent seaward winds in moving it back again seaward.

Above high tide the sand is arrested by stones, grasses, trees or other obstructions; the various grasses and shrubs which grow just landward of high water being very effective sand collectors, cause the formation of sand

FIG. 9. HEAVY WAVES STRIKING AGAINST VERTICAL SEA WALLS

The views show clearly the inefficiency of vertical sea walls and their effect of shore erosion. During gales the waves hit the face of the wall and the falling water scours out material from the front of the wall, hastening its collapse

dunes. Dunes may also be formed by wind action alone; but on the coast margin the formation of irregularly shaped dunes is chiefly due to vegetable life and obstructions such as sea-formed sand embankments.

The shape of sand dunes (Fig. 7) when not interfered with is much the same as that of sand ripples. The windward (seaward) slope is usually from 4½ to 10 deg. with the horizontal, and the leeward (landward) slope any angle up to the angle of repose of sand. On the coasts of New Jersey and Long Island the dunes commonly vary in height from 10 to 40 ft.

Sand dunes, unless fixed either naturally or artificially by vegetation, move inland in much the same way as sand ripples. The velocity of inland travel is the rate of advance of the crest, which takes place by the accumulation of sand on the steep leeward slope *BC* (Fig. 7). The supply is brought by the wind in two ways—first, by rolling of coarse sand grains up the windward slope *AB* and over the crest *B*; and, second, by the deposit of part of the flying sand caught by the eddy on the

[10] "Transactions," American Society of Civil Engineers, Vol. 23.

leeward side and not tossed away again. Experience has shown that the rate of inland travel of unfixed dunes varies from 3 to 100 ft. per annum.

In places where the sand blown in from the foreshore is not replaced by sand brought by the sea, erosion of the foreshore naturally takes place; the inland movement of the sand dunes being followed by the inland movement of high-water line. Reference to Fig. 7 shows that when the crest has been moved from B to B' the high-water line will have moved from A to A'. While considerable attention has been given to the important effects of waves and currents in causing erosion, little attention has been paid to the also important effects of wind action in causing erosion of the New Jersey and Long Island coasts.

In some places sand dunes which became naturally clothed with vegetation, and were therefore fixed, have subsequently moved inland at a rapid rate owing to the destruction by man of the vegetation of the dunes. The advantage of vegetation for protecting sand dunes and preventing their inland movement was recognized in the United States in early days. On Dec. 16, 1758, an act was passed to prevent the destruction of the vegetation on the sand dunes. It prohibited farmers from turning cattle out to graze on the dunes of Long Island and other parts of the Atlantic Coast where they formed a natural protection against coast erosion. This act was renewed by state law in 1789.

Sand-dune embankments are also moved inland by wave action during gales and very high tides, material being moved by the waves up the windward slope and deposited on the leeward slope, thus causing a gradual inland movement of the sand ridge. Near Barnegat inlet and at other places, especially where the dunes are low, they are overflowed by the waves during gales.

All along the New Jersey coast evidence is to be found that sand-dune embankments and sand islands have moved inland a considerable distance. At Squan Beach, near Loveladies Island Life Saving Station, north of Beach Haven, Brigantine Beach, Peck's Beach and many other places salt marsh turf is found exposed on the foreshore at low tide, in which can be seen the imprints of cows' and horses' hoofs. The salt marsh was undoubtedly formed at the back of the sand dunes and under their shelter, and the dunes have moved inland and encroached on the salt marshes.

The rate at which the sand spits and sand islands move inland is difficult to determine with accuracy. In the "Annual Report of the New Jersey State Geologist for 1885" it is stated that "the lateral movement has in many cases amounted to more than the breadth of the beach since the settlement of the state, and is at present (1885) going on with undiminished activity." Captain Allgor stated that at Island Beach the westward movement had taken place at the rate of about 1.05 ft. per annum for the 40 years previous to 1885. When a sand spit is built up across a bay from the mainland on one side to the mainland on the other, then it will be moved inland at the same rate as the inland movement of the high-water line due to erosion of the mainland on either side.

In addition to causing coast erosion the inland movement of sand dunes in some places also causes the devastation of large areas of fertile land, and the dunes in their inland advance have buried houses, and even villages of considerable size. There are some places on the New Jersey coast where orchards have been buried within the lifetime of their owners and timbered lowlands have in several cases been overwhelmed and the timber destroyed by the encroaching sand.

Erosion Caused by Groins

Harbor piers and jetties projecting into deep water have practically the same effect as headlands and promontories in checking the littoral drift, and causing accumulation on the windward side and erosion on the leeward side.

Badly designed high groins constructed on the foreshore for coast-protection purposes in many cases stop littoral drift and cause a serious danger instead of a protection, owing to the great scour which takes place to the lee side of such structures. Fig. 8 shows a plan of a high groin erected across the foreshore. Material moved by oblique wave action is trapped on the corner at B. The shore material moved away from the leeward shore at DE is, therefore, not compensated for by littoral drift from the windward side of the groin and erosion must result. As material is scoured out from DE, material from CD moves into the hollow and erosion therefore takes place from C to E. After the high groin is full up on one side a water-falling action takes place. When waves roll obliquely up the shore on the windward side AB and the water falls over to leeward, stirring up material and scouring out the foreshore between C and D, and in course of time undermining the groin.

Erosion Caused by Sea Walls

Badly designed sea walls and bulkheads are also a very frequent cause of serious erosion. Waves striking the face of a vertical or nearly vertical wall project great volumes of water skyward, which, falling at the back of the wall, will scour out the backing, and falling at the toe will certainly dredge out the material in front of the foundations and cause the structure to collapse (Fig. 9). Even without the actual projection, the backwash of a wave from a curved smooth surface has a powerful scouring action.

Artificial Removal of Shore Material

The artificial removal of foreshore material for building and other purposes is also a cause of sea encroachment. If the foreshore is lying at its normal level, neither erosion nor accumulation going on, and material is removed from any part, an immediate result is to cause a gradient sloping to that part from all around it, and a consequent feeding in of material from the adjoining foreshore to fill the place of that which has been removed, so that the removal of material from a foreshore belonging to one person may prejudicially effect his neighbor's property.

Erosion of the coasts of New Jersey and Long Island cannot be traced to any one cause. There are, as has been pointed out, many forces and agencies at work on the coast line tending to cause erosion, and it is only by a close study of the conditions prevailing on any particular shore that engineers with experience in coast works can trace the cause or arrive at any conclusion as to the best means of coast protection.

Reinforced-Concrete Tower Tank at Middleborough, Mass.

By George A. Sampson*

SYNOPSIS—*A 500,000-gal. tank, 41 ft. in diameter and 60 ft. high, supported on a cylindrical tower; total height, 165 ft. Tank has a hemispherical bottom. Details of design and specifications.*

The application of reinforced concrete to standpipes and elevated tanks for the storage of water covers a period of only twelve years, during which time about thirty of particular note have been constructed. Some engineers consider its use purely experimental as yet and have grave doubts about the adaptability of concrete to such work, while a few unequivocally condemn its use in such structures of any considerable size. Further progress in transforming the field of experiment to one of recognized general practice can be accomplished only by a thorough discussion and study of the design, construction and success or failure of similar structures. With this end in view, an outline of the principal features of the design of the Middleborough tower tank is herewith presented.

Middleborough Water-Works

The supply is surface ground water collected in a large open well, aërated and filtered for the removal of iron and manganese, and pumped to a 230,000-gal. steel standpipe 20 ft. in diameter by 103 ft. in total height, situated in a residential portion of the town. The pressure is sufficient for domestic uses, but entirely inadequate for fire protection, which is now furnished by direct pumping. Under the present laws of Massachusetts, this requires the employment of six men at the pumping station, three of whom must be licensed engineers.

After careful inspections, in 1911 and 1913, it was recommended that the steel standpipe built in 1885 be discontinued as somewhat unsafe. After due consideration of the various features of the problem, it was decided to build a structure of adequate capacity and with the storage at a sufficient elevation to meet the demands of the combined domestic and fire service without resorting to direct pumping. The present population supplied with water is 5000, and the average daily consumption is 350,000 gal. Allowing for future growth, assuming that the pumps would be operated only by day and supposing that a large fire should occur in the early morning, demanding eight effective fire streams for two hours, the desired storage capacity was fixed at 500,000 gal.

The most available location for a new structure was on Barden Hill, about one mile southeast of the center of the town and 35 ft. above the present site. To furnish suitable fire-fighting facilities at the business center of the town, with due regard for the hydraulic factors involved, demanded that 386,000 gal. of this quantity be stored at a height of 121 ft. above the ground at Barden Hill, or 53 ft. higher than the top of the existing standpipe.

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Types of Structure Considered

Four types of structure were considered: First, a steel standpipe about 40 ft. in diameter by 160 ft. high, which would have a total capacity of 1,500,000 gal.; without engineering this was estimated to cost $44,000; second, a concrete standpipe of the same general dimensions and capacity, at a cost of $39,000; third, a steel tank of 500,000 gal. supported on a steel tower about 110 ft. high, at $22,000; and fourth, a concrete tank of 500,000 gal. capacity, supported on a concrete tower and estimated at $24,000.

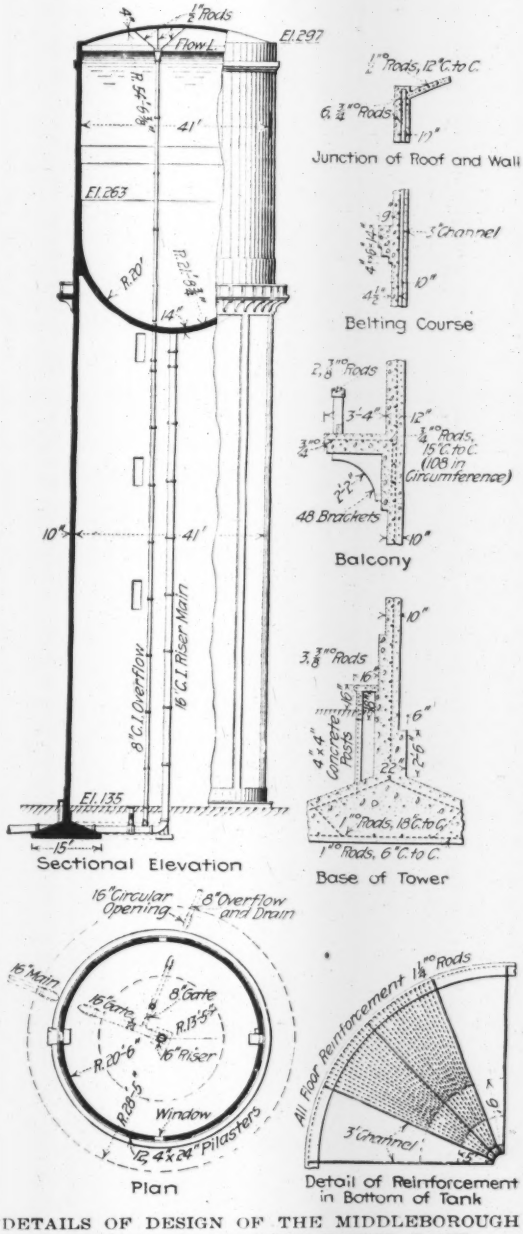
The first type was not further considered on account of its cost and large maintenance expense; nor was the second, because of its cost and the impracticability of constructing a satisfactory concrete structure subject to a 160-ft. head of water. The decision rested then between a steel and a concrete tower-tank. For comparison it was assumed, as an average period, that a steel structure would require two coats of paint inside and outside every five years, and if so treated would have a life of thirty years. The annual cost of painting would amount to $340. Also direct pumping into the mains for several days during the time of painting would be necessary. This should be avoided if possible, especially in view of the growth of the town. For preserving the quality of the water, maintaining it at a more uniform and desirable temperature and adorning the landscape, the concrete tank was preferable. In first cost, the steel tank was about $2000 less, and possessed the advantage of being a common type of structure.

With the foregoing and other considerations in mind, the one question to be decided was: Can a concrete tank be constructed that shall remain sufficiently watertight at all times to prevent unsightly discoloration and disintegration from seepage and leakage, combined with freezing temperatures? The method of supporting the tank on a concrete tower was of secondary importance and was considered capable of a reasonable solution. After an exhaustive study of existing concrete standpipes and tower tanks, together with the apparent causes contributing to their good and bad features, it seemed a conservative conclusion that a reinforced-concrete tower tank of 500,000 gal. capacity could be built so as to warrant its use.

Design of the Tank

In the design the question of the foundation was the first to be considered. The soil upon which the footing will rest is extremely fine, well-compacted sand, overlaid for about one-quarter of the area with a thin bed of gravel. The safe bearing power of this fine material was determined experimentally by loading a timber set at the elevation of the bottom of the footing. At a loading of from 1 to 3½ tons per sq.ft., the settlement was uniform and slight, being only $\frac{1}{32}$ in. for each additional ½ ton; but 4 tons per sq.ft. produced a relatively large

settlement—$\frac{5}{32}$ in. for the last ½ ton. The maximum loading was therefore fixed at 31½ tons per sq.ft. The total dead-weight on the foundation soil is 4485 tons, or 2.28 tons per sq.ft. The wind load was determined from an assumed velocity of 70 mi. per hr., producing a wind pressure of 30 lb. per sq.ft. on the horizontal projection of the cylinder, thereby adding 0.68 tons per sq.ft. with a full tank, and producing a total maximum pressure of 2.96 tons. Although the wind pressure is slightly



DETAILS OF DESIGN OF THE MIDDLEBOROUGH REINFORCED-CONCRETE TOWER TANK

more than doubled with an empty tank, it is more than offset by the reduction in the dead-weight of the water, so that the maximum mentioned represents the worst condition.

A cylindrical tower was adopted in preference to a number of columns, because with it a more ready and definite computation of stresses was possible; while it could be made to present a more pleasing appearance, was better adapted to the contours at the site, and seemed more economical to construct. The plain aspect of a simple cylinder was relieved by the addition of twelve 4x24-in. pilasters which were also assumed to take their proportion of the load. The appearance of solidity at the base is accomplished by the use of a concrete seat supported on the foundation to prevent settlement. The compression at the bottom of the 10-in. tower wall resulting from the dead load is 483 lb. per sq.in., which is increased to a maximum of 631 lb. by a wind of 70 mi. per hr.

Details of the Tank Proper

The tank proper is 41 ft. in inside diameter and 64 ft. deep from under side of the dome (El. 300) to the inside of the hemispherical bottom (El. 236), the dome having a rise of 4 ft. and the bottom a depth of 20 ft. The straight walls of this tank vary in thickness from 10 to 16 in. and are reinforced to care for increasing pressure toward the bottom. The bottom varies in thickness from 14 in. at the lowest part to 18 in. at the connection to the tank walls. Its reinforcement is given in the detail in the accompanying figure.

By far the most important part of the structure and the most difficult to design and construct is the tank proper. The writer wishes to emphasize the use of a hemispherical bottom, which should prove superior to the flat beam and slab construction commonly employed for small elevated tanks, and to the dome-bottom used in a few instances for tanks of considerable size. The advantages of the hemispherical over the dome design lie in a 17-per cent. increase in capacity for the same height of vertical wall, or a 34-per cent. decrease in the height of wall for the same capacity; a practically self-cleaning bottom; a better distribution of stresses because the enormous thrust in the dome bottom, which is concentrated at the bottom of the tank, the most critical point, is replaced by a uniform tensile stress coincident with and analogous to the tension in the tank wall. The only other stress at the junction between wall and bottom is a vertical downward load which is easily calculated and cared for. The reduction in height of vertical wall is of prime importance in reducing the chances of leakage. The hemispherical bottom will be poured continuously to make a monolithic bowl up to the point where it becomes a part of the supporting tower. The cost of storage should be no greater than for the dome bottom.

The thickness of concrete at and near the bottom of the wall of the tank and in the hemispherical portion is such that without any assistance from the steel reinforcement its stress in tension is about 250 lb. per sq.in. The tensile stress in the steel, acting independently, would be approximately 14,000 lb. per sq.in.; and with both materials acting in conjunction, the computed tension in the concrete is 215 lb. and in the steel 2150 lb.

The tank is covered to guard against the growth of algæ not uncommon in filtered water exposed to the light, and to prevent freezing. The roof is a dome with its thrust carried by steel reinforcement embedded in the top of the tank wall.

The connection between tank and water main is through a 16-in. flanged riser pipe fitted with an expansion joint. An 8-in. flanged overflow, also with an expansion joint, discharges at a safe distance from the foundation.

The essential qualities of the cement required is fineness of grinding, strength and a slow initial set. The sand, in addition to the usual requirements, shall show a tensile strength equal to standard Ottawa sand, and for the tank it may be necessary to mix sands from two or more sources to obtain the proper gradation of sizes. The coarse aggregate for the tank is gravel varying in diameter from ¼ in. to 1¼ in., and, if necessary, screened into various sizes and combined in the proportion deemed expedient to secure maximum density. Either gravel or broken stone may be used for the other parts of the structure.

The steel reinforcement is hard grade round rods, plain or deformed, and conforms to the specifications for concrete reinforcement bars adopted by the Association of American Steel Manufacturers in 1910, and revised in 1912. The laps are at least 40 diameters, and in addition thereto each joint in the tank is secured by two "Crosby" wire clips, or their equal. Vertical channels are employed to support the horizontal reinforcement of the tank, drilled to the exact spacing of the rods and provided with suitable means for rigidly holding them.

The proportions of concrete for the foundation are 1:2.5:5; for the tower, 1:2:4; for the hemispherical bottom and wall of the tank to El. 279, 1:1:2; from El. 279 to 293, 1:1.5:3, and above El. 293, including the roof, 1:2:4. The relative amounts of sand and stone may be varied, the percentage of cement to the sum of the aggregate to remain as stated.

Details of the Construction

The concrete must be mixed thoroughly and by an approved machine. Care must be taken in placing to prevent the separation of the ingredients, and for the wall of the tank concreting will be carried on each successive day. No special waterproofing ingredient is required, nor is a metal dam being used at the junction between days' work. All laitance is to be removed from the surface at the end of the day's work before the initial set occurs. The old surface is to be cleaned with a wire brush and, immediately before placing new concrete, coated with a neat cement grout and a layer of mortar. It is hoped that it will not be necessary to resort to any waterproofing lining or coating on the interior.

Steel forms designed and made by the Blaw Steel Construction Co. are being used for the outside surface of the tower and tank. The height of each ring is 4 ft.; and at least two complete rings are required to insure the placing of concrete on successive days. No permanent bolts, wires or other ties that would be cast in the concrete are being used to secure either the steel or wooden forms.

The steel forms are producing a smooth, uniform surface which will require very little finishing. The exposed base of the tower below El. 139 will be rubbed to such surface with a carborundum block and water.

The following clause in regard to water-tightness may be of interest:

> The concrete in the bottom and wall of the tank shall be substantially water-tight at all times. Any leakage amounting to jets or visible seepage shall be repaired by and at the expense of the contractor and by methods approved by the engineer. Small damp spots, if few in number, which do not increase, nor disfigure the appearance of the concrete, will be considered to meet the requirements hereunder. It is intended that the work shall be sufficiently waterproof that severe freezing will not at any future time threaten the integrity of the concrete, and that the general appearance of the structure will not suffer through efflorescence or other disfiguring stains.

The contract for the work has been awarded to the Hennebique Construction Co., of New York, for $23,140, and the structure is now under construction. Alvin C. Howes is chairman of the Water Commissioners and Thomas F. Dorsey is Resident Engineer for the writer, who is responsible for the design and construction.

Provo Reservoir Company's Irrigation Project

By Clarence S. Jarvis*

The Provo Reservoir Co. was organized in 1908 to construct a high-line canal 40 mi. long from Provo River, capable of supplying water to some of the most desirable lands in Utah and Salt Lake valleys.

Beside filings for 150 sec.-ft. of the surplus water in Provo River, which is ordinarily available until after

FIG. 1. TRIAL LAKE, UNION RESERVOIR CO.
Used jointly by a number of irrigation companies. View taken from top of dam. Note the ice sheet in the distance

the middle of June, appropriations were made for storing flood water in more than thirty lakes and meadows near the head of Provo River. Other irrigation companies are jointly interested in the larger reservoirs (through the Union Reservoir Co.), the maximum area of a single storage basin being about 140 acres for a 40-ft. depth of

FIG. 2. A 30-FT. RESERVOIR OUTLET CUT

storage. The lake basins (Fig. 1 is an example) appear to be pockets formed by glacial action. Outlet cuts (Fig. 2) through the terminal or lateral moraine that forms the rim of each basin provide means of drawing the lake surface down to the desired depth. Short rock-fill dams

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(Fig. 3) inclosing a core of puddled clay, mountain soil and rock waste are built where necessary.

These reservoir sites are situated at El. 9500 to 10,500 above sea level. Due to the remoteness from inhabited districts, the high altitude, the consequent inclement weather and short seasons for construction, and the rocky and rugged character of this locality, construction has proceeded more slowly than was expected; but the reservoirs already built have demonstrated the value of this storage system and its efficiency. The water when released from the reservoirs flows down the natural channel of the Provo River a distance of about 60 mi. to the intake of the Provo Reservoir Co.'s canal; and with the 40 mi. of canal the delivery will be through 100 mi. of waterway.

The canal takes out of Provo River about 1 mi. above Olmstead in Provo Cañon. Thence it proceeds through a 6x7-ft. tunnel for 1000 ft. through solid limestone; thence through 2100 ft. of No. 144 Maginnis steel flume, a 200-ft. section of which has recently been replaced by a 6-ft. corrugated ingot-iron pipe over a mud slide; thence through a 260-ft. rock tunnel; thence 2600 ft. in a 60-in. redwood

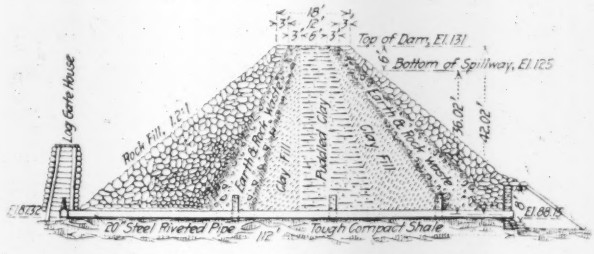


FIG. 3. EARTH AND ROCK-FILL DAM

The 20-in. steel outlet pipe was made from ¼-in. plates. The sections were 20 ft. long, joined by ⅝x2½-in. steel angle flanges and lead gaskets

stave pipe used as an inverted siphon for crossing Provo River, and delivering on Provo Bench.

To cross Dry Creek, north of Lehi, the water is delivered through 1000 ft. of wood-stave pipe of 49-in. diameter, with a maximum pressure head of 80 ft. on the inverted siphon.

At Jordan Narrows a riveted steel pipe, 1458 ft. long, 40 in. in diameter, has been constructed out of ¼-in. and $\frac{5}{16}$-in. steel plate. The approach to this pipe is a reinforced-concrete conduit 1700 ft. in length and approximately 48 in. in diameter. The maximum pressure head that the steel pipe may have to withstand is 270 ft.

The gradient used in tunnels, flume and pipe lines ranges from 0.002 to 0.004; and in the canal from 0.0005 to 0.002, dependent upon the required cross-section and the type of soil encountered.

The canal system has delivered in excess of 100 sec.-ft. of water satisfactorily to the lands from Provo Bench to the Bull River Canal north of Lehi Junction. It is intended ultimately to deliver in excess of 170 sec.-ft. through this canal system, about half of it to be carried into the West Jordan district, in Salt Lake County.

The lands to be directly benefited are about 8000 acres in Utah County, and the same area in Salt Lake County. As these tracts include some of the most fruitful and desirable lands in this district, well situated for markets and along railroad lines, the project has decided advantages. However, it might be well to point out some of the chief difficulties, which are as follows:

1. The actual cost of storing water in such reservoir basins as are included in this reservoir system ranges from $6 to $20 per acre-foot.

2. The patrolling of stored water from the head of the river for a distance of 60 mi., passing numerous headgates and dams in the lower valleys that have been accustomed to diverting the entire flow of the river during the low-water season, has been rendered more difficult through lack of definite data on loss in transit, the varying flow of the river, the primitive measuring devices and the crude regulating structures in use in some of the canals; and from an innate prejudice felt by owners of old water rights against the new applicants that cause a change in the regulations of the river.

3. Acquiring right-of-way for more than 10 mi. across improved farm and city property, through orchards, between houses and barns, where the right-of-way far exceeded the cost of construction, and where numerous bridge, culvert and flume structures were necessary along the canal line.

4. The definition of the term "surplus water." Some authorities hold that all water in a river that is not being applied to beneficial and economical use is surplus and subject to appropriation, even though a careless or wasteful use of water might consume the entire flow of the stream. Upon this definition depends in large measure the success or failure of a new irrigation project. Efforts are being made to reach a compromise agreement, providing that when all old rights are satisfied on a basis of 50 or 60 acres per second-foot of water, then all of the surplus above this amount belongs to the Provo Reservoir Co. until its acreage is supplied on a basis of 70 acres per second-foot; after which all further surplus shall be divided pro rata according to the acreage and the needs of the various classes of soil.

Throughout the construction of this project Joseph R. Murdock has been president and manager of the company; C. S. Jarvis has been Chief Engineer, and E. D. Clyde has been Superintendent of Construction on the reservoirs. I. H. Jacob is Resident Engineer, and has had charge of water distribution since 1912.

The Production of Radium on a manufacturing basis by the U. S. Bureau of Mines from Colorado carnotite ore has recently been announced by F. K. Lane, Secretary of the Interior. The Bureau has been studying the production of radium with a view to decreasing the cost. The figure reached is $36,050 per gram of radium bromide and this includes the cost of ore, the insurance, repairs and depreciation on plant, the development cost of the process and all expenses. Radium bromide has been selling for $120,000 to $160,000 per gram. This reduction in cost does not mean an immedate drop in selling price, since the carnotite claims are closely held. Ten claims are being operated at Long Park, Colo., by the National Radium Institute, with which the bureau has been coöperating. A concentrating plant for low-grade ores has been erected by the Institute. Grinding and sampling machinery and an extraction plant have been set up in Denver, the present capacity being for three tons of ore per day. The average recovery has been 85 per cent., and with the last five carloads of ore it has been 90 per cent.

United States Public Lands unappropriated and unreserved amounted on July 1, 1915, to 279,000,000 acres. Of this area 173,000,000 acres has been surveyed and 106,000,000 acres is unsurveyed. These public lands belonging to the Federal Government are in 25 states. The bulk of the land, however, is in the 10 arid states. Nevada heads the list with 55,000,000 acres, and then follow in the order named Arizona, Utah, Wyoming, New Mexico, California, Montana, Colorado, Idaho and Oregon. Oregon's public lands amount to 15,442,000 acres. South Dakota has nearly 3,000,000 acres, and Washington a little over 1,000,000 acres. In most of the other states where public lands are located the amount is comparatively small.

Paving Problems of Queensboro Bridge, New York

A number of difficulties have arisen in maintaining the wood-block pavement on the Queensboro (Blackwell's Island) Bridge, New York City, some of which are peculiar to the bridge structure, but others throw some light on the wood-block paving problem in general. The bulging, "volcanic" blowups exhibited in some other wood-block pavements have been particularly pronounced on the main spans and steel-supported sections of the Queens approach.

The original construction of the bridge-floor system consisted of longitudinal buckle-plates, 5 ft. wide between stringers from 12 to 20 ft. long between floor-beams or cross-frames, with from two to four buckles to each plate. The buckle-plates were not stiffened. On this floor was placed a solid concrete base from 1½ to 4½ in. thick. On the concrete was placed a cushion or bedding course of ½ in. of dry sand and cement to receive the pavement of 4-in. creosoted wood blocks.

No great trouble was experienced with this pavement for about two years, but with increased traffic the buckle-plates deflected and vibrated to such an extent under heavy loads that the concrete base was badly broken up. Where the base did remain intact the sand-mortar cushion, having no drainage, was continually damp and after every rainstorm was completely saturated. When these storms were followed by sudden changes in temperature the blocks heaved and blew up to an alarming degree, sometimes forming "pimples" in the roadway 3 ft. high.

The roadway is 52 ft. 3 in. wide, with a car track on each side, and no longitudinal expansion joints were provided along the rails. In this great width of pavement the expansion and contraction of the steel-floor system and the loose unhardened cushion course provided a constantly shifting base for the blocks, and further aggravated the pavement troubles. On one or two occasions vehicle traffic over the bridge had to follow the street-car tracks to avoid the humps and billows.

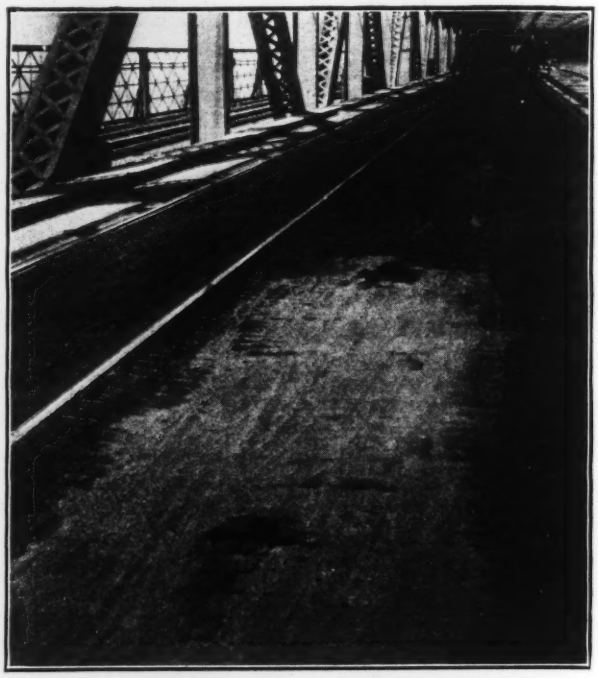


FIG. 1. OLD QUEENSBORO BRIDGE PAVEMENT, SHOWING BREAKS

In the fall of 1912 Edward A. Byrne, then Assistant Chief Engineer of the Department of Bridges of New York (now Acting Chief Engineer), designed a new pavement foundation, which has since been experimented

FIG. 2. LOOKING ACROSS QUEENSBORO BRIDGE DURING PAVEMENT RELAYING

Buckle-plates stripped of concrete in foreground; drain pipes in place; Z-bar and angle-iron reinforcement; new concrete foundation in background

with and found successful. A considerable stretch of the bridge floor has been paved by this new method, and work is now in progress on other sections of the bridge, as the accompanying illustrations show.

The first consideration was that no appreciable dead load be added to the bridge. It was essential, however, to stiffen the buckle-plates by riveting a 3x3x½-in. angle across the fillets of each buckle-plate (see Fig. 2). On the Queens approach spans the angles are put on the upper side of the buckle-plates, so that their top edges are flush with the top of the new concrete foundation, but on the main span 3½x3½x½-in. angles are riveted to the bottoms of the buckle-plates.

The tops of the steel stringers carrying the buckle-plates are capped with 3x12-in. yellow-pine planks, treated with carbolineum. The planks are dressed to a uniform thickness of 2½ in., laid flat and bolted through the flanges of the stringers by machine bolts placed 30 in. apart. The heads of the bolts are countersunk to be flush with the top of the plank.

Every two panels, averaging about 50 ft., a transverse $6x\frac{9}{16}$-in. Z-bar is riveted to the flanges of the floor-beams. This projects 3 in. above the top of the new concrete foundation, so that two perpendicular faces are provided for the paving blocks on either side to bear against. These Z-bars divide the pavement into distinct sections, and prevent the creeping of the blocks which occurs in the old method of construction. The new pavement is in longitudinal strips 15 ft. wide, each strip separated by an expansion joint.

The buckle-plates are first cleaned of all the old concrete foundation and repainted. The drain holes are then cleaned out, and 6-in. sections of iron pipe reaching just below the top of the new concrete foundation are inserted. The pipes are held in place by first filling a little concrete around them in the buckles. Wooden plugs are inserted to keep the inside of the pipe clear of concrete.

A new 1:2:4 concrete foundation is then placed with its surface flush with the tops of the 3x12-in. planks. Strips of wood about ½x1 in. are laid on the green concrete from the center pipe to the upgrade sides of the buckle-plates. When these are removed after the concrete has set, shallow drainage gutters are left. On this foundation 3-in. creosoted wood blocks are laid without a cushion or bedding course, and with sand-filled joints.

Mr. Byrne states that the blocks, resting partly on the wood stringer caps, and partly on the concrete, might reasonably be expected to splinter to some extent, but this has not been serious, and there has been no heaving of the blocks in the new pavements. There are 34,600 sq.yd. of pavement on the buckle-plates, and as yet on account of lack of funds but a small percentage has been relaid as described. The cost has varied from $4 per sq.yd. for the first experimental sections to $3, exclusive of reinforcing buckle-plates.

The traffic on Queensboro Bridge has become extremely heavy. The average daily traffic for two July days was:

AUTOMOBILES

Trucks	Delivery	Pleasure	Totals
1172	692	6312	8176

HORSE-DRAWN VEHICLES

One-Horse	Two-Horse	Three- and Four-Horse	
782	868	60	1710
Total for one day			9886

Such a heavy, fast-moving traffic is unusually severe on any pavement, but the difficulties encountered are laid chiefly to the foundation conditions.

New Specifications for Rails; Pennsylvania R.R. System

Revised specifications for 100-lb. rails of carbon steel have been adopted by the Pennsylvania R.R. system (including the lines east and west of Pittsburgh). The chemical requirements are given herewith. A special feature is the requirement for chemical tests of finished openhearth rails (after passing the physical tests). The purpose of this is to guard against the segregation of the metal, which has been discovered in so many of the rails that have broken in the track.

The following paragraphs are given direct from the specifications, with their original numbers attached. Certain unimportant paragraphs are omitted.

CHEMICAL COMPOSITION OF RAIL STEEL

4. The chemical composition of the rails rolled from each melt of steel, determined as described in Section 6, shall be within the following limits:

Elements	Bessemer Process	Openhearth Process
Carbon	0.45 to 0.55%	0.60 to 0.75%
Phosphorus	not to exceed 0.10%	not to exceed 0.04%
Manganese	0.80 to 1.10%	0.60 to 0.90%
Silicon	0.05 to 0.20%	0.10 to 0.30%

Note 1—In the event of nickel and chromium being present to the extent of 1% and 0.35% respectively, these elements will be considered as the equivalent of 0.07% of carbon in the above requirements.

Note 2—When the analyses for carbon by the mill chemists and by the railroad chemists do not agree, a tolerance of two points below the minimum or two points above the maximum will be allowed to cover such variation before condemnation.

5. It is desired that the percentage of carbon in an entire order of rails shall average as high as the mean percentage between the upper and lower limits specified.

ANALYSES OF RAIL STEEL

6. In order to ascertain whether the chemical composition is in accordance with the requirements, analyses shall be furnished as follows:

(a) For bessemer process, the manufacturer shall furnish to the inspector, daily, carbon determination for each melt before the rails are shipped, and two chemical analyses every 24 hr., representing the average of the elements, carbon, manganese, silicon, phosphorus and sulphur, contained in the steel, one for each day and night turn respectively. The analyses shall be made on drillings taken from the ladle test ingot not less than ⅛ in. beneath the surface.

(b) For openhearth process, the makers shall furnish the inspectors with a chemical analysis of the elements, carbon, manganese, silicon, phosphorus and sulphur, for each melt.

(c) For openhearth process, a check analysis will be made by the purchaser of a piece of rail representing a melt, after the rails from that melt have passed the physical requirements. On request of the inspector, and in his presence, the manufacturer shall furnish from one of the drop-test pieces representing the melt, drillings satisfactory to the inspector, taken with a ⅝-in. flat drill, parallel to the axis of the rail, at a point one-third of the distance from the upper corner to the center of the head [designated as point "O"]. The analysis from these drillings shall conform to the chemical requirements specified in Section 4, and failure to meet these requirements shall be sufficient cause for the rejection of the entire melt.

(d) For openhearth process, after the rail has passed the physical requirements, additional drillings will be taken from the same rail, and in the same manner as specified in Section 6 (c), at the junction of the head and web [designated as point "M"]. The carbon determination from these drillings (Note 2, Sec. 4) shall be within 12 per cent. of the amount found at location "O." If the test from the top rail fails to meet this requirement, all the top rails from the melt shall be rejected, and a similar determination shall be made from location "M" of a second rail. If this test fails, all the second rail from the melt shall be rejected, and a similar determination shall be made from location "M" of a third rail. If this test fails, all the remaining rails from the melt shall be rejected.

(e) If, however, the segregation found at location "M" in any rail in a rolling exceeds 25 per cent., when determined as provided for in 6 (d), the progressive testing of the second and third rails will not be permitted on any subsequent melts; but on such melts the failure of the top rail to pass the requirements provided for in 6 (d) will cause the rejection of the entire heat.

Expansion Joints in Granite-Block Pavement

The remarkably smooth grouted granite-block pavements of Worcester, Mass., were described in detail in *Engineering News* of Oct. 22, 1914. Some of these pavements are now 14 yr. old and practically nothing has been spent for real maintenance, most of the expenditures being for repairs to openings in the pavements. The chief difficulty has been that the pavements have outlived the street-car tracks, and many of the rail joints, especially in Main St., have been in very poor condition for some time past.

When at last compelled to renew its track, the street railway company strenuously objected to replacing grouted granite blocks over its roadbed, because of the difficulty in making track repairs in this kind of a pavement; but the city eventually won its contention, and the street is being restored to its original condition. While a grouted granite-block pavement is admittedly difficult to open for track repairs, this is chiefly an argument for building better track structures.

FIG. 1. BLOCKS FROM OLD PAVEMENT

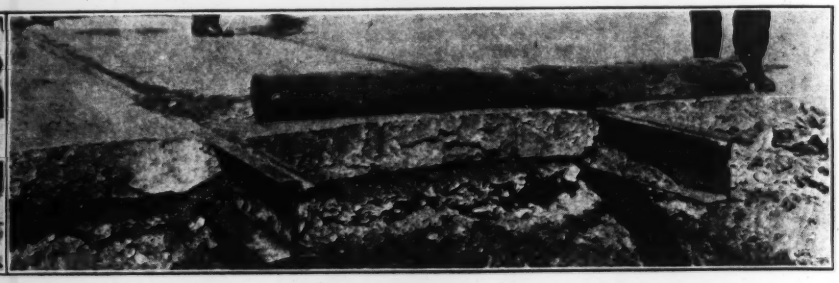


FIG. 2. MONOLITHIC STRUCTURE OF OLD GRANITE-BLOCK PAVEMENT

The monolithic condition of these pavements is illustrated by Figs. 1 and 2. Many blocks were broken through as easily as they could be broken apart at the joints. About 50 per cent. of the blocks were reclaimed by chipping off the grout by hand, the cost of which was about 1c. per block. The broken blocks were crushed to make concrete aggregate for the foundation of the new pavement, at a saving of about 80c. per ton over other material.

Expansion Joints

Two innovations are being introduced in relaying the pavement. Expansion joints separating the track pavement from the rest of the street pavement are being tried in both the foundation and block courses. Fig. 4 shows the track preparatory to placing the foundation concrete with a Barrett specification expansion joint about ½ in. thick by 6 in. wide, nailed to the ends of the ties. The concrete base is 6 in. thick.

Concrete is not being placed under the ties, which, however, are gravel ballasted; the foundation is of uniform thickness across the entire width of street. The ties used are not treated with preservatives. The old 95-lb. girder rails are being replaced with 125-lb. grooved girder rails.

The expansion joint in the pavement surface is shown in Fig. 3. This is of the same character as the joint in the foundation and is put a block-width from each outside rail of the double track. The course of blocks next the outside rail are laid lengthwise, as shown in the illustration. The blocks keyed into the old pavement have broken joints of at least 2 in.

FIG 3. EXPANSION JOINT IN GRANITE-BLOCK SURFACE

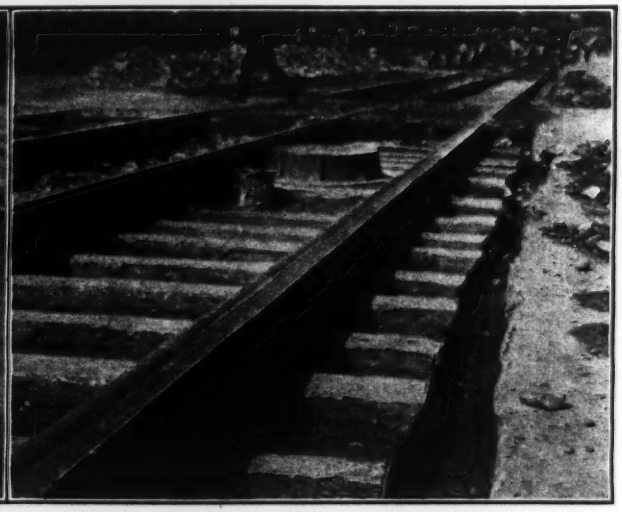


FIG. 4. EXPANSION JOINT IN CONCRETE FOUNDATION

The purpose of these expansion joints primarily is to prevent the rumbling noise caused by the heavy suburban street cars. It is expected that the expansion joints will permit the track to vibrate under the cars without affecting the rest of the pavement. That the rumbling was

noticeable in the old track was probably because the monolithic pavement was carrying a part of the load on the tracks, which it would seem could also have been eliminated by a more rigid track foundation.

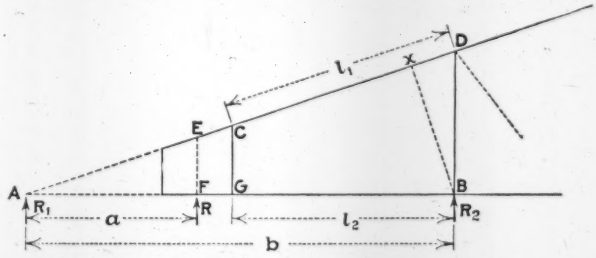
The track pavement work is being done by F. A. Wilkerson, Superintendent of Tracks, Worcester Consolidated Street Ry., in conformity with the city's specifications. The city engineer of Worcester is F. A. McClure.

Moments at Eccentric Heel-Joint of Roof Truss

By Edward Godfrey*

It appears that the solutions proposed in *Engineering News* of May 20, 1915, p. 990, for the problem of the heel-joint of roof trusses, suggested by the writer, Apr. 22, p. 783, are to go unchallenged and an approximately correct solution is not forthcoming. This is a very important problem in structural engineering.

The solutions given by the correspondents are all wrong in the most important item of the problem, namely the magnitude of the bending moment taken by the chord members. Nor did any of them say anything about finding the direct stress in the chord members which is another important part of the problem. Furthermore, they all



seem to consider the chord members as pin-ended at the second panel point, whereas they are continuous.

The bending moment is given as Ra (see the sketch herewith). This is manifestly incorrect, for it would mean that the moment in the chords for a given shape of truss would increase uniformly as R is moved toward the position R_2, whereas the moment must be zero for the position R_2. The fact is the moment in the chords as R moves to the right is a maximum where a equals ½ of b, and it describes a parabola as R moves from A to B.

Part of this problem is to find the direct stresses in the chords. This is a simple operation by the method of moments, but in a graphic diagram trouble is apt to be met with in closing the polygon. As the reaction R does not strike the intersection of the chords, some method must be devised of substituting members or forces that will intersect. This could be done by assuming members BE or DF, but the direct stresses of the chords would not then be correct.

If a section be cut vertically through the truss between EF and DB it will be seen that the shear in this section is equal to R. Part of this shear is taken up in the vertical component of the stress in the top chord ED. The remainder of the shear will be taken up by the two members ED and FB in bending, and the amount of the bending moment will be dependent upon the horizontal distance between CG and DB. It will thus be seen that this horizontal distance has much to do with the bending moments, though the published solutions referred to consider only the distance a.

To find the bending moment in the chords it is necessary to find the difference between R and the shear in ED due to direct stress. By moments

$$\text{Direct stress in } ED = \frac{R(b-a)}{BX}$$

$$\text{Shear in } ED = \text{stress} \times \frac{DB}{AD} = \frac{R(b-a)\,DB}{BX \,.\, AD}$$

Substituting for DB/BX its equivalent AD/b, we have

$$\text{Shear in } ED = \frac{R(b-a)}{b} \qquad (1)$$

It will be seen that this shear is just what would be the left-hand reaction of a beam AB, hence the remainder or what would be the right-hand reaction of a beam AB passes to the right through the chords in transverse shear producing bending, and the next step in the problem is to distribute this shear according to the rigidity or stiffness, not only at C and G, but also at D and B. The chords are more rigidly held at C and G than at D and B, but they are merely continuous at all of these points and no great error is entailed in considering C and D equally rigid, as well as G and B.

If the top and bottom chords are of the same section it would be close enough to consider all four points C, D, B and G equally rigid, with the bending moment

$$M = \tfrac{1}{2} R \frac{a}{b} \times \frac{l_2}{2} = \frac{Ral_2}{4b} \qquad (2)$$

If the chords are of different sections, and if the lengths l_1 and l_2 be taken into consideration, as they should in an exact solution, the stiffness of the two chords will be directly as their respective moments of inertia and inversely as the cubes of l_1 and l_2. If I_1 and I_2 represent the moments of inertia of the top and bottom chords respectively, and if M_1 and M_2 represent the bending moments in the top chord (at C and D) and in the bottom chord (at G and B) respectively, we have

$$\frac{M_1}{M_2} = \frac{I_1 l_2^3}{I_2 l_1^3} \qquad (3)$$

But

$$2M_1 + 2M_2 = \frac{Ral_2}{b} \qquad (4)$$

Solving

$$M_2 = \frac{Ral_2^2}{2b\left(1 + \frac{I_1 l_2^3}{I_2 l_1^3}\right)} \qquad (5)$$

Also M_1 is found to be an expression of similar form, with I_1 and l_1 replacing I_2 and l_2 respectively.

If member DB is not vertical the problem is more complicated. A close approximation may be arrived at by using the mean between l_2 and the horizontal projection of l_1 in place of l_2 in eq. (2) and (4).

In many cases the top chord will be so much stiffer than the bottom chord that the full moment could be assumed as taken by that chord. The moment would then be Ra/b times one-half of the horizontal projection of l_1.

In order to work out the stresses in the graphic diagram the truss should be considered as extended to A and instead of using R the two forces R_1 and R_2 should be used. These are the reactions at A and B that would be found for a beam of span b loaded with the force R.

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Report on Life of Wood Pipe*

By D. C. Henny†

SYNOPSIS—Results of an investigation of the lasting qualities of wood pipe carrying water. Made under instructions from the director and chief engineer of the Reclamation Service. A life of from 12 to 25 years is shown where conditions allow complete saturation of the wood.

This inquiry on the life of wood pipe covered three general types—continuous wood-stave, section wood-stave and sectional bored. The information contained is tabulated according to the following classification: Table 1, continuous-stave, uncoated fir. Table 2, continuous-stave, uncoated redwood. Table 3, continuous-stave and wire-wound, coated fir. Table 4, continuous-stave and uncoated Texas, Colorado and yellow pine.

The term "fir" is applied to the most common wood of the Pacific Northwest. Scientifically it is a bastard spruce, first named by Douglas. Locally it is known as fir or Douglas fir, and distinction is sometimes made between yellow and red fir. These come from the same kind of tree, the yellow fir being most common. The large trees yield yellow fir, while the wood from smaller trees is apt to show a reddish tint. Sun exposure also appears to affect the color to some extent. In California the wood is termed Oregon pine. This wood is strong, and can be obtained in long lengths. Mill run of commercial sizes probably averages over 20 ft. in length. It contains pitch, which forms in seams and pockets, normally occurring even in commercially clear wood within certain limits. The trees usually yield less than 25 per cent. of wood free from sap and knots. The expense of eliminating these defects has led to the practice, as regards pipe construction, of permitting some small and sound knots and sap.

Redwood grows on the Pacific coast from Santa Cruz north into southern Oregon. By reason of the average large diameter of the trees, the logs are cut shorter than in the case of fir, and the commercial mill run averages about 16 ft. in length. The wood is not as strong as fir. About 50 per cent. of the wood cuts up clear and free of

*From the August issue of the "Reclamation Record" of the United States Reclamation Service.

†Consulting Engineer, United States Reclamation Service, Spalding Building, Portland, Ore.

TABLE I. CONTINUOUS-STAVE FIR PIPE, UNCOATED

Location	Length, mi.	Diameter, in.	Pressure, ft.	Soil	Built	Remarks	Life	Serial No.
Astoria, Ore.	7.5	18	0–150	Loam and sand with decaying vegetable matter	1895	Gradually increasing leakage due to decay of wood Replaced by redwood pipe in 1911	16	1
Basin, Wyo.	. .	40–96	.....	Not stated	1906–1908	In good condition except where not constantly full	8+	2
Belle Fourche, S. D.	0.3	36	.....	Clay and gumbo	1912	In good condition	3+	3
	0.4	42						
Clarkson, Wash.	7.0	48	.....	Volcanic soil, also loose rock	1907	Where in loose soil or rock, decayed near top; 400 ft. replaced in 1911; 1000 ft. replaced since	4	4
	4.0	40						
	1.5	32						
						Balance O. K.	8+	
Everett, Wash.	. .	20	50–125	Clay and loose soil	Before 1909	Decayed except where buried in clay Replaced 2600 ft.	6 6+	5
Gothenburg, Neb.	0.2	72	0–46	Loose, sandy, partly uncovered by wind	1890–1891	Decayed, especially where uncovered, causing increasing leakage; new wood pipe built in 1913, covered 2 ft. and uncoated.	22	6
Logan, Utah	1.2	60	15–80	Not buried	1904	Decayed at pitch seams, otherwise O. K.	11+	7
Loveland, Col.	8.0	12	0–200	1–3 ft. cover	1902	Some decay on top of pipe	13+	8
Ogden, Utah	5.0	72	.....	.....	1897	Small amount at upper end decayed where pressure light and soil very porous; no decay where exposed	18+	9
Post Falls, Idaho	5.0	42	0–40	Loamy gravel, loose	1910	1 mi. with maximum pressure badly decayed Remainder partly decayed	5 5+	10
Preston, Utah	0.6	60	0–300	Not covered	1905	I. good condition except leaking in places due to settlement	10+	11
Pueblo, Colo.	. .	48	.....	Adobe	1906	Commenced leaking, due to decay, in 1909. Serious decay in 1913 had gone one-half through wood. Then incased in concrete	7	12
Seattle, Wash.	0.5	54	.....	Clay, sand, gravel	1899–1900	Part has decayed and has been reconstructed Superintendent's estimate:		
	13.5	44				In sand and gravel	7–12	13
	8.5	32				In clay	12–20	
						Pipe buried in clay is reported as good as new	16+	
Sugar City, Colo.	4.0	.....	.....	Soil not stated, cover 2-4 ft.	1904	Wood not decayed; ½-in. bands corroded; replaced in 1910 with ⅝-in. iron	11+	14
Sunnyside, Wash.	2.1	55	25–130	Volcanic ash, gravel, sand, about ⅔ laid on ground not covered	1908	Uncovered portion badly decayed in 1913		
	0.3	48	130		1908	Uncovered and coated 1913-14 Relaid 1914–15:		
						In open soil	6	15
						In air or water	7+	
Utah power pipes	1.6	36	.....	On saddles	1903–1904	Decayed	12	16
	2.1	36	6–226	In open trench	1907	In good condition	8+	17
	3.1	58	8–180	Laid on earth	1910	In good condition	5+	18
	1.5	30	4–91	In open trench	1908	In good condition	7+	19
	4.7	30	12–13	On earth	1910	No decay	5+	20
	1.8	24	10–37	On earth	1907	Wood from old circular flume, built in 1901, in fair condition	14+	21
	2.7	78	24–44	On earth	1913	Wood in good condition	2+	22
	4.3	132		On concrete saddles	1914	Wood in good condition	1+	23
	0.8	102	15–125	On wooden saddles	1908	Some decay at butt joints	7+	24
	1.3	74	.....	Buried	1909	Some decay where back fill porous	6+	25
	1.2	24	.....	On earth	1908	In good condition	7+	26
Wenatchee, Wash.	1.7	44	*	Buried in clayey soil except where crossing bridge	1903	The upper end for 750 ft. did not run full at all times, and was replaced in 1908, and was again replaced later		
						Balance of pipe, where buried, O. K. so far as known	12 (?)	
						Pipe where exposed in good condition	12+	26a
	2.3	42	*	Buried in clayey soil except where crossing bridge	1907	Buried pipe decayed where no pressure, renewed in 1914		
						Balance O. K.	8—	26b

* 0 to about 100 ft.

sap, which has led to the custom of demanding freedom from knots and sap in ordinary pipe specifications.

Continuous-stave pipe consists of staves breaking joint and clamped together with bands, which since 1888 have universally been made up of steel bolts and lugs, the latter being made of cast iron, malleable iron, or pressed steel. The butt joints are made tight by a metal or wooden plate.

Wire-wound or machine-banded pipe is pipe made up in sections from 10 to 24 ft. in length. It consists of staves held together by galvanized wire, spirally wound around the pipe under heavy tension. The wood is usually kiln dried, which is not the case with continuous-stave pipe. The ends of each section are machine turned to an exact diameter, sometimes slightly tapered. Sections are united by some form of sleeve, a sleeve made up of staves and wound around with wire or individual bands being most common.

Wire-wound pipes for light pressures are often provided with mortise and tenon ends, dispensing with the necessity of sleeves. Under the head of wire-wound pipe in the tables a few cases have been grouped of bored-log pipe, which is wound not with wire, but with flat iron or steel. Such pipe is often jointed by metal sleeves, generally riveted iron or steel.

The wood surface in continuous-stave pipe is, with a few exceptions, uncoated, while sectional pipe is heavily coated, the coat consisting of asphaltum and tar, with sawdust on the outer skin.

The investigation has had in view especially the life of pipe as affected by the durability of the wood. The life as given in the tables refers to the wood alone, and is further confined as to sectional pipe to wood in the pipe sections and not to wood in the sleeves. Figures followed by the plus sign show age rather than life. The information collected is based mostly on reports received from managers or owners, and in small part on personal observation. It is not as complete as is desirable, which, however, is not due to failure to elicit further information.

Reviewing the information as grouped under its headings, it may be estimated that under conditions of continuous water pressure the life of various kinds of pipe may be as follows:

Wood	Condition	Years	Serial No.
Fir	Uncoated, buried in tight soil	20	13
Fir	Uncoated, buried in loose soil	4–7	4, 10, 12
Fir	Uncoated, in air	12–20	9, 10, 21
Redwood	Uncoated, buried in tight soil, loam or sand, and gravel	Over 25	27, 29, 38, 41, 48
Fir	Well coated, buried in tight soil	25	63, 64, 75
Fir	Well coated, buried in loose soil	15–20	51, 52, 69, 70

Under conditions interfering with complete saturation of the wood, the life is cut down materially. These conditions are in the case of uncoated pipe brought about by open soil and low water pressure. The effect of coating appears to be equivalent to tight soil cover, with possibly the additional advantage of tar in the coating acting as a disinfectant. The effect of unfavorable conditions as to lack of complete saturation is serious in redwood, resulting in a life which may be shorter than 15 years

TABLE 2. CONTINUOUS-STAVE REDWOOD PIPE, UNCOATED

Location	Length, mi.	Diameter, in.	Pressure, ft.	Soil	Built	Condition	Life	Serial No.
Butte, Mont.	9.1	24	0–150	Loam, mixed with gravel and loose rock	1892	Sound except some deterioration where covered with loose rock only	23+	27
	6.2	26	0–150	Loam, mixed with gravel and loose rock	1899	Sound except in few places where pressure is light	15+	28
	8.5	24	0–150	Loam, mixed with gravel and loose rock	1900	Sound except in few places where pressure is light	15+	28
Denver, Colo.	16.4	.....	.....	Covered	1890	Still in use	25+	29
Greensboro, N. C.	*	.....	.....	Soil not stated				
	1.3	18	20–200	Covered	1907	Sound, both wood and bands; superintendent would use nothing else	8+	30
	3.0	20	20–200	Soil not stated				
	2.5	24	20–200					
Hollywood, Calif.	0.3	12	50	Loam	1898	Much of wood decayed in 1910, probably due to light pressure and dry soil. Possibly not full at all times. (Finkle.) Company gives far better report	12	31
	6.9	14	50					
Ithaca, N. Y.	†	†	†	Not stated	1905	No maintenance cost; so far as known in good condition	10+	32
Logan, Utah	1.5	18	0–70	Gravelly soil	1893	In excellent condition; wood sound; metal slightly rusted	22+	33
Los Alamitos, Calif.	3.5	18	30–50	Not stated	1906	Used 4 mo. each year for disposing of waste from sugar factory; wood slightly deteriorated. Most bands replaced	.....	.....
Mount Nebo, Utah	0.3	48	0–70	Not stated	1895	Wood sound; no corrosion in bands reported	20+	34
	1.1	‡72	0	On wooden saddles	1895	Wood slightly decayed. Bands rusted; some replaced	20+	35
North Yakima, Wash.	0.2	32	0–90	Heavy soil partly exposed on rocky side hill	1894	Wood decayed some at upper ends of siphon; bands somewhat rusted	21+	36
	0.2	32	0–90	Heavy soil partly exposed on rocky side hill	1903	Wood sound; bands slightly rusted; replaced in 1914 by larger pipe in connection with concreting of canal	12+	37
Ogden, Utah	5.5	24	.....	Not stated	1890	4674 ft. light pressure replaced in 1911 but still used as overflow. Balance still in use in good preservation. Where exposed and protected from sun by board wood perfectly sound	25+	38
Palo Alto, Calif.	1.4	12	0	Salt marsh	1900	Used as outfall sewer; runs one-half to one-third full; bands badly corroded. Replaced in 1908 on 3000 ft.; replaced in 1911 on 1025 ft.	15+	39
Poso, Calif.	0.03	72	10	Sand	1893	Used for irrigation; full part of each year; reported in fair condition	22+	40
Provo, Utah	5	14	0–125	Clay, gravel, light loam	1891	In light loam, nearly gone; in gravel, fair; in clay, sound	24+	41
	0.5	60	.....	Clavey soil except 500 ft. on bridge	1910	Staves show decay at ends where many joints leaking and where not covered. No water for 4 mo. in year	5+	42
Redlands, Calif.	0.4	48	Light	Partly in earth; partly on trestle	1892	Decayed considerably where in contact with earth; was replaced with ditch as a matter of precaution in 1912	20+	43
Temple, Tex.	1.7	18	5–35	Not stated	1899	In good condition as to both wood and bands; only trouble where wood damaged by sharp rocks thrown in for back fill	16+	44
Utah Power & Light.	2.1	26	6–144	Exposed; on earth	1910	In good condition	5+	45
	1.1	28	5–176	Exposed; on earth	1907	In good condition; staves show tendency to buckle up at joints	8+	46
	1.7	22	3–137	Buried	1907	In good condition so far as known; no maintenance expenses	8+	47
Yuma, Ark.	2.1	26	90–120	Adobe, sand, gravel	1892	In good condition; has replaced a few staves; superintendent says as good as 20 years ago	23+	48

* Partly Texas pine. † Not stated. ‡ One-half circumference of flume.

TABLE 3. FIR PIPE, CONTINUOUS-STAVE AND WIRE-WOUND TYPES, COATED

Location	Kind of Pipe	Length, mi.	Diameter, in.	Pressure, ft.	Soil	Built	Character of Coat	Condition	Life	Serial No.
Belle Fourche, S. Dakota	C.-S.	0.3	60	0–20	Clay and gumbo	1914	2 coats asphalt compound	Wood, sound	1+	49
Butte, Mont.	W.-W.	4.4	14	0–300	All kinds soil	1911	Mfrs.' standard	Good condition	4+	50
	W.-W.	*	*			†1895	Mfrs.' standard	Good condition	20+	51
Clarkston, Wash.	W.-W.	*	*	60–300	Volcanic soil	†1903	Mfrs.' standard	Good condition, except wood in some collars is soft and spongy, losing strength and causing leakage. Some decay in 4-in. size attributed to sap	12+	52
Colfax, Wash.	W.-W.	14.0	8–10	0–200		1904	Mfrs.' standard	Good condition, except decay and leakage in upper section, also 1500 ft. section over Gerber Hill, where pressure is light; upper section not now used	11+	53
								Gerber Hill section replaced by same kind pipe laid on lower grade in 1910	6	54
Greeley, Colo.	W.-W.	38.0	20	0–205	Not stated	1907	Mfrs.' standard	Good condition; 6 staves decayed presumably account sap. Some collars decayed where pressure light	8—	55
Hood River, Ore.	W.-W.	5.6	2–20	10–150	Loam	†1905	Mfrs.' standard	Wood sound; few leaks in joints	10+	56
Lewiston, Idaho	W.-W.	*	*	100–300	Not stated	1906	Mfrs.' standard	Good condition, except collars, which in some instances have decayed	9+	57
Montrose, Colo.	W.-W.	*	*		Strongly alkaline	1905	Mfrs.' standard	Wood in fair condition; some staves in pipe decayed where pressure light after 5 years; decay in collars first noted after 5 years; chief difficulty, erosion of wire, being replaced by endless number of clamps	10+	58
North Platte, Neb.	C.-S.	0.4	24		Light, sandy loam	1914	Tar, 2 coats, 1–16-in.	Good condition	1+	59
	W.-W.	0.3	18		Light, sandy loam	1913	Mfrs.' standard	Good condition	2+	60
	W.-W.	0.1	16			1913	Mfrs.' standard	Good condition	2+	61
North Yakima, Wash	W.-W.		4–20	10–200	Loam and gravel	†1905	Mfrs.' standard	Wood in pipe sound; wood in some collars spongy and decaying; some leakage in joints	10+	62
Olympia, Wash.	W.-W.	10.0	2–12	100–220	Clay soil with some gravel in places	1896–1912	Mfrs.' standard	In excellent condition, including wire	19+	63
	Bored-log	4.5	2–8	100–200	Same	1890–1900		In good condition, including flat iron hoops	25+	64
Post Falls, Idaho	W.-W.	5.0	12–24	0–50	Gravelly clay open	1910	Mfrs.' standard	In good condition except decaying collars	5+	65
Provo, Utah	W.-W.	2.5	21	‡	Gravelly	1907	Mfrs.' standard	Decayed; sleeves replaced with concrete; requires rebuilding	8	66
	W.-W.	2.6	18–24	§	Gravelly	1911	Mfrs.' standard	Some decay in pipe; heavy maintenance account decay in collars	2	67
Pueblo, Colo.	C.-S.	3.0	24	||	Adobe, loam, and shale	1904	Probably manufacturers' standard	Wood so far as known all right; wire began to fail in 1911; 4,000 ft. replaced in 1912; remainder in 1913	8+	68
South Bellingham, Wash.	C.-S.	....	20	0–200	Clay and loose soil same as in Bellingham	1900	Asphalt	In excellent condition	15+	69
Spokane, Wash.	W.-W.	*	*	**	Gravelly soil	....		After 5 years begins to trouble in seams and joints, not from decay, but water ram	....	....
Sunnyside, Wash.	Bored-log	*	*	0	Volcanic ash, gravel, etc., buried 4 to 8 ft.	1893	Mfrs.' standard	Main canal turnouts taken up in 1905–6 because too small, pipe found in excellent condition	13+	70
	Bored-log	0.4	13	3–15	Volcanic ash, 1.5 to 2 ft. cover	1903	Mfrs.' standard	Sound, tight, metal little corroded	12+	71
	W.-W.	0.4	4	10–60	Volcanic ash	1898	Thinly painted with creosote	Decayed in 4 years	4	72
	W.-W.	....	24			1906		Sound, but extensive repairs on wooden collars	9+	73
	W.-W.	....	14	0–25	Sandy loam	1908	Mfrs.' standard	Sound, except 1000 ft. at upper end, had not run full; lasted 3 years, renewed in 1912; balance gives trouble with leaky joints; otherwise will probably last 10 to 20 years	7+	74
Tooele, Utah	Bored-log	....	....	....	Buried	1890	Probably manufacturers' standard	Wood in good condition where kept wet; pipe has flat iron hoops	25+	75

* Various sizes. † And later. ‡ Light pressure. § Heavy pressure. || Not stated. ** Water-works pressure.

(No. 31), but is much more marked in fir, where it may be as low, in spite of coating, as six years (No. 54).

Shortened Life

It is quite possible that the life of individual pipe is not merely affected by surrounding conditions, but may have been shortened by the quality of the individual pieces or the treatment of the wood. The wood from the bottom of a tree is close grained and likely to last longer than from the top of the tree, even though free from sap. Bastard-sawn staves, especially in fir, resist penetration of the water in the pipe to a greater extent than diagonal- or vertical-grained wood and may show a shorter life. Again, too rapid kiln drying, which may at times have been practiced on lumber from which staves for wire-wound pipe were run, may have partially destroyed the vitality of the wood.

In this investigation it has not been found practicable to differentiate between the causes which in this manner may have shortened the life of the wood in pipe,

TABLE 4. VARIOUS WOODS, CONTINUOUS-STAVE PIPE, UNCOATED

Location	Wood	Length, mi.	Diameter, in.	Pressure, ft.	Soil	Built	Condition	Life	Serial No.
Denver, Colo.	Colorado pine	....	....	....	Not stated	1884	Top staves above water line decayed rapidly; part near river in wet soil still in use in 1912	18+	76
	Yellow pine	7.7	24, 30, 37	....	Not stated	1886–1887	Some lasted till 1907, when replaced with fir	....	....
	Texas pine	16.4	30	....	Not stated	1890	Still in use	25+	77
Pueblo, Colo.	Texas pine	....	*28	....	Adobe	1900	Condition in 1913: Decayed irregularly one-half to three-fourths of stave; some new staves account decay; bands near leaks or in moist soil corroded rapidly	23+	78
Pueblo, Colo.	Texas pine	....	†54	....	Not stated	1890	Coated part with tar, part with asphalt; 1500 ft. running partly full, replaced in 1895 with redwood	5	
							Portion painted with asphalt replaced in 1910	20	79
							Portion painted with tar sound in 1910	20+	

*Partly redwood. † Coated.

nor to ascertain what has been the effect of permitting some sap in the wood from which decay may have spread to the adjoining sound wood. Unquestionably much wire-wound pipe has been manufactured without being subjected to strict supervision and inspection.

Conclusions

On the whole the following conclusions seem justified as regards the relative life of fir and redwood in pipe:

(*a*) Under favorable conditions of complete saturation fir well coated may have the same life as redwood uncoated.

(*b*) Either kind of pipe will have a longer life if well buried in tight soil than if exposed to the atmosphere. Such life may be very long, 30 years and over, if a high, steady pressure is maintained.

(*c*) Either kind of pipe will have a longer life if exposed to the atmosphere than if buried in open soil, such as sand and gravel and volcanic ash, provided in hot and dry climate it be shaded from the sun.

(*d*) Under questionable conditions, such as light pressure or partially filled pipe, fir, even if well coated, may have only from one-third to one-half the life of redwood.

(*e*) Under light pressure the use of bastard-sawn staves in fir pipe should be avoided.

(*f*) The use of wooden sleeves in connection with wire-wound pipe is objectionable and has caused endless trouble and expense. It is possible that the objection may be partially overcome by dipping ends of sleeves in creosote and by applying a heavy coating of tar to the ends of the sleeves. Saturation of sleeve wood will never be as perfect, however, as of the straight pipe, and full creosote treatment of the wood or else some form of metal sleeve, either riveted iron or steel heavily coated, or cast iron will probably be well worth the extra cost.

(*g*) If wooden sleeves are employed, they should be provided for sizes at least from 10 in. up with individual bands to permit taking up leaks.

Special Comments

Pitch seams do not occur in redwood. In fir they should be distinctly limited as to size, frequency, and depth. In respect to knots, there appears to be no reason for making any distinction between the two classes of wood. Small, sound knots, if not passing through the full thickness and not occurring close to edges or ends might be permitted in either wood. Sap is objectionable, and the higher cost in prohibiting it entirely or of putting narrow limitations on it is probably justified in both fir and redwood.

Wood pipe is not suitable in cases where it can not be kept full and under pressure during periods of use. Coating can not under such condition be expected to afford protection against decay. Coating should be continuous and heavy, not less than one-sixteenth inch to be fully effective, and should preferably consist of more than one individual coat of a mixture of asphaltum and tar, or of an application of gas tar followed by one or more applications of refined coal tar. Little experience, however, can be quoted in support of all-tar coating.

All tentative conclusions herein presented are necessarily based on the limited data which could be collected and will undoubtedly require amendment as further experience accumulates.

Long-Term Rainfall Records in New York State

By Edward H. Sargent*

That we are still in a "trough" of a cycle of rainfall is shown by the accompanying hydrograph of rainfall in New York State. The diagram represents the mean of the progressive ten-year averages of the precipitation from 1829 to 1914 inclusive at Albany, Rochester and New York City, the rainfall records at these stations constituting the only long-term continuous records in New York State.

The method used in computing and plotting the hydrograph was as follows: The average rainfall for the first ten years (1829-1838) was computed for each of the three stations and the mean of these averages computed and plotted as for the year 1834. These ten-year periods were then arranged successively, each beginning and ending one year later than the preceding.

The hydrograph shows a well-defined cycle of increasing and decreasing precipitation. The three stations used, Albany, Rochester and New York City, are so distributed

RAINFALL AT ALBANY, ROCHESTER AND NEW YORK
Mean of progressive 10-year averages of rainfall, 1829 to 1914, inclusive

that they should give a fair average of rainfall conditions over the entire state during this period. This is checked by the fact that the average rainfall in New York State as planimetered from the long-term isohyetal map is 38.80 in., while the average for the above-cited three stations is 38.14 in.

The mean annual precipitation for the first and last halves of the period covered by the hydrograph for each station was as follows:

	Albany	Rochester	New York City	Average
1829-1871	40.33	32.94	43.00	38.76
1872-1914	35.36	33.59	43.63	37.53

From this tabulation it is seen that at Rochester and New York City the mean rainfall is greater for the last half of the period than for the first half, while the reverse is the case at Albany.

Coal Consumption in Brazil—Brazil is a tropical country; coal for power purposes, therefore, is practically the only source for which there is any demand. Of this the greater portion is consumed by the various steamship lines. The railways also require a certain amount. In this connection it is interesting to note that one of the railways has installed several oil-burning locomotives. Factory requirements are not large, as electric power generated hydraulically is relatively cheap. Until the European war started, Cardiff and Newcastle were the prevailing types of coal used. In unloading from steamers coal is delivered on the dock by the steamer's derrick and then worked by hand. In coaling ships the coal is carried by laborers in baskets; this method is considered the most economical, labor being cheap. The average cost of unloading or loading is stated by Consul General A. L. M. Gottschalk, Rio de Janeiro, to be 50 to 60c. per ton.

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Piledriving Destroys a Tunnel by Clay Pressure

An 8-ft. brick tunnel crossing under the Cuyahoga River on the line of Clark Ave., Cleveland, Ohio, was completely crushed by lateral pressure and flow of the clay subsoil produced by piledriving for bridge piers within a few feet of the tunnel. This remarkable accident required complete rebuilding of a tunnel for a total length of some 200 ft.—about 100 ft. at each of the two river piers.

The tunnel contains a 4-ft. steel riveted water-main seated centrally in the 8-ft. tunnel on chairs. It was built about five years ago. The Clark Ave. viaduct, which the city is now building, has its center line about 25 ft. south of the line of the tunnel. The piers carrying the river span have substructures 66x96 ft., thus extending over the tunnel. The bottom of concrete is at El. —29; the top of the tunnel is about El. —46, or only 17 ft. below the base of the piers. Although the piers rest directly on the stiff clay subsoil of the Cuyahoga Valley and adjacent uplands, the city bridge engineer, F. D. Richards, considered it desirable to drive piles to get reliably solid bearing. This required piles 40 to 50 ft. long. As piles of such length could not be driven in the tunnel area, it was decided to keep the long piles 15 ft. apart, or 2½ ft. clear of each side of the tunnel brickwork (see sketch Fig. 2), and cover the intervening space with short piles whose points would be 3 or 4 ft. above the tunnel. Some reinforcing steel was placed in the bottom of the pier over this space to help distribute the weight uniformly among the entire pile group.

The short piles over the tunnel were driven first; then the driving of the long piles was begun with the rows nearest the tunnel in order to progress outward on both sides so that any crowding action of the clay would act away from the tunnel. By the time the whole footing had been piled (938 piles), water in the tunnel shafts indicated that something was wrong with the tunnel. An examination showed so serious a condition of affairs that it was necessary to shut off the water main; and after the completion of the piledriving in both piers the tunnel had to be rebuilt in these sections.

Under the east pier the tunnel was totally destroyed. Its sides had been forced in on the whole width of the pier (about 70 ft. along the tunnel), and the clay and broken brickwork filled nearly the entire cross-section around the pipe (see Fig. 1). The pipe was lifted off its chairs for 100 ft. or more each way. Nowhere had a pile actually touched the tunnel; the damage was caused wholly by the crowding pressure of the clay. At the west pier the tunnel was not crushed in this way, except for local damage at one point where one pile came close to the shell. However, on the width of the pier the tunnel was lifted from its original position by a maximum amount of from 2½ to 3 ft. To reconstruct the tunnel, the clay and brickwork were removed by carrying forward a circular drift, to the line of the outside of the new brickwork plus the thickness of a solid timber lining of 12-in. segment. This was done in short sections and the timbering kept close. When the drift was forward far enough to expose an entire length of the pipe this was unbolted and removed, and thereupon the corresponding length of new brickwork was laid up inside the timber lining. The same procedure was followed through the whole damaged section. At each end where the connection with the old brickwork was made, care was taken to tooth in as well as possible. While it might be thought from the nature of the accident that the clay is plastic, it is on the contrary very hard and firm. This was shown some months earlier, in tearing out the old west shaft of the tunnel, which being within the area of the west pier had to be replaced by a shaft 100 ft. farther west. On removing the brick-

FIG. 1. INTERIOR OF CRUSHED CLARK AVE. TUNNEL UNDER EAST PIER OF VIADUCT

During reconstruction. Segmental timber lining in foreground. Candle on original center line of tunnel

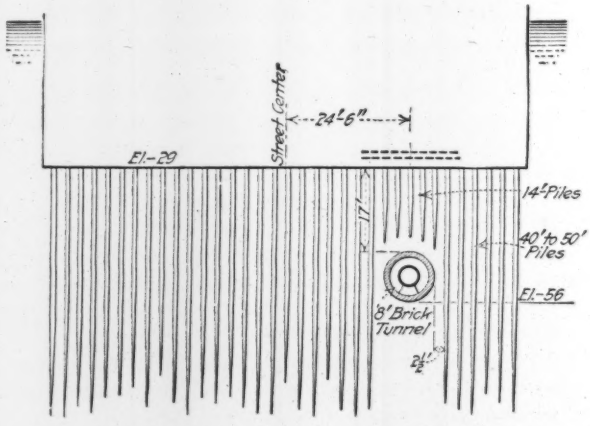


FIG. 2. SKETCH OF PIER AND TUNNEL ON SECTION TRANSVERSE TO CLARK AVE.

work of the old shaft the clay back of the lining was found to stand clear of the lining and to show the mattock marks made when it was built.

A. B. Cook, resident engineer under Mr. Richards on the viaduct construction, had supervision over the tunnel repair work also. The contract for this was let to the Manson Co., Engineers' Building, Cleveland, Ohio. The work cost about $20,000, the aggregate length of the two rebuilt sections being 180 ft.

Rapid Tunnel Driving Under the Bonus System

By J. R. McFarland*

The rapid advance made on some of the recent "record" tunnels, notably the Mount Royal, Arizona Copper, St. Louis Water-Works, Laramie Poudre, and Los Angeles Aqueduct, seems to have been due very largely to the bonus system of wages. This system was used at all the tunnels mentioned, and is the only practice of note that was common to those five contracts. Different types of drills, some with and some without carriage mountings, both long and short rounds, and different methods of handling the muck were used.

The cost of tunnel driving is practically constant for each day's work, but the cost per foot varies with the rate of progress. From past performances the average advance per day, week and month is known, and it is expected that under normal conditions this rate of progress will be maintained. If exceeded, there is of course a considerable saving to the contractor; and where the bonus system is in force he pays his men a certain amount, as their just portion of the saving effected, for each linear foot of advance in excess of the base rate. The saving realized by the bonus system in the labor cost is shown in the tabulation which follows:

LABOR SAVING BY BONUS SYSTEM

	Per Cent.
Mount Royal Tunnel	15
Arizona Copper Tunnel	20
St. Louis Water-Works Tunnel	50
Laramie Poudre Tunnel	31

The cost of driving the tunnels during the record months under the bonus system has been compared with the cost of driving before the bonus system was adopted. Before considering the several projects separately the accompanying tables should be examined. Table 1 gives some general data as to character of rock, size of heading, etc., while Table 2 shows the approximate number of men at work.

TABLE 1. DATA OF TUNNELS WHOSE RECORDS ARE COMPARED

	Mount Royal Tunnel	Arizona Copper Tunnel	St. Louis Water-Works Tunnel	Laramie Poudre Tunnel
Rock	Trento limestone with igneous dikes	Granite porphyry; good drilling	Both hard and soft limestone	Moderately hard, gray or red granite
Size of tunnel heading	8x12 ft.	8x8 ft.	8 ft. diameter (circular)	7½x9½ ft. (elliptical)
Average cross-section	3.56 cu.yd. per ft. advance	70.04 sq.ft.	95 sq.ft.	62 sq.ft.
Record advance in 31 days	810 ft.	799 ft.	745 ft.	653 ft.

TABLE 2. TOTAL LABOR ON TUNNEL CONSTRUCTION

	Mount Royal Tunnel	Arizona Copper Tunnel	St. Louis Water-Works Tunnel	Laramie Poudre Tunnel
Superintendents	2	..	1	1
Foremen or bosses	3	1	3	3
Drillers and helpers	24	12	24	15
Muckers	24	15	22	18
Miscellaneous labor of all classes	59	12	51	22

*5710 Kimbark Ave., Chicago, Ill.

The bonus paid to the men at the Mount Royal tunnel was 7% of their daily pay for every foot over 14 ft. per day. The influence of the bonus system on the work at this tunnel is shown by Table 3.

TABLE 3. MOUNT ROYAL TUNNEL

Costs	A Typical Month Without Bonus	Record Month With Bonus
Labor	$9,989.00	$15,883.00
Equipment and supplies	2,927.00	4,750.00
Total, including overhead	13,916.00	21,633.00
Advance, at 14 ft. per day	434.00	810.00
Per foot (labor)	$23.00	$19.61
Per foot (labor, equipment and supplies)	29.76	25.47
Per foot (total)	32.06	26.70
Per cubic yard (labor)	6.48	5.51
Per cubic yard (total)	9.00	7.51
Saving (in labor) in cost per foot, by bonus system	....	3.39
Saving (in labor, equipment and supplies) per foot	....	4.29
Saving (total) per foot	....	5.36
Saving in total cost per cubic yard	....	1.50

At the Arizona Copper tunnel the men received regular wages, and for every foot of advance over 350 ft. per month a bonus of $7 was divided. This was only a trial, but although it was found that the bonus was excessive, it was not reduced, as the duration of the test was short. The tunnel was driven the first month without a bonus and the three succeeding months with a bonus. A comparison of the advances and the costs per foot of tunnel in each case is shown by the figures in Table 4.

TABLE 4. ARIZONA COPPER TUNNEL

	Without Bonus	With Bonus		
Costs	First Month	Second Month	Third Month	Fourth Month
Labor	$3773.00	$5314.24	$5802.69	$6574.77
Equipment and supplies	2013.33	2013.33	2013.33	2013.33
Labor, equipment and supplies	5786.33	7327.57	7816.02	8588.10
Advance in feet	350	582	664	762
Per foot (labor)	$10.78	$9.13	$8.74	$8.63
Per foot (labor, equipment and supplies)	16.53	12.59	11.77	11.27
Cubic yards of rock removed, labor, equipment and supplies	6.33	4.82	4.31	4.32
Saving on labor, per foot, by bonus system	....	$1.65	$2.04	$2.15
Saving on labor, equipment and supplies, per foot	....	3.94	4.76	5.26
Saving on labor, equipment and supplies, per cubic yard	....	1.51	1.82	2.01

Three shifts per day were employed at the St. Louis Water-Works tunnel. A minimum advance of 72 ft. per week was set, and for all over that amount the following bonus rates were paid: Superintendent, 50c. per ft.; heading foreman, 50c.; muck foreman, 30c.; drill runners, 30c.; drill helpers, 20c.; nipper, 20c.; muckers and cage men, 10c. Each shift was credited with one-third of the extra progress and received one-third of the bonus.

TABLE 5. ST. LOUIS WATER-WORKS TUNNEL

Costs	Estimated Cost per Day Without Bonus	With Bonus
Labor	$381.75	$381.75
Total bonus per day	.....	66.80
Advance, feet per week	72	745
Labor per foot	$37.09	$18.66
Labor per cubic yard rock removed	10.54	5.30
Saving in labor, per foot, by bonus	.....	18.43
Saving in labor, per cubic yard, by bonus	.....	5.24

Three shifts per day were employed on the Laramie Poudre tunnel. For the first part of the work each underground workman was paid a bonus of 25c. per day for each 25 ft. in excess of 400 ft. per month. This system was soon discontinued, as it was found to be not only cumbersome but excessively high, and the following system was adopted: When the rate of driving for any calendar month exceeded 400 ft. and was less than 500 ft., each underground employee was paid $10 extra; between 500 and 600 ft. the bonus was $15, and between 600 and 700 ft.

it was $20. In order to distinguish the bonus money from the regular wages the bonus was paid by check. The itemized account is given in Table 6.

TABLE 6. LARAMIE POUDRE TUNNEL

Costs	Estimated Cost per Day Without Bonus	With Bonus
Straight wages	$242 65	$242 65
Total bonus per day		29 68
Advance per month in feet	400	653
Labor, per foot	$18 76	$12 94
Labor per cubic yard removed	8 16	5 60
Saving per foot due to bonus		5.82
Saving per cubic yard due to bonus		2 56

The bonus system at the Los Angeles Aqueduct tunnels proved to be a great success, and all tunnels here were driven under this system. The bonus was shared by the tunnel foreman, shiftboss, miners and muckers. All back-trimming was done by the crew, which shared the bonus. Overbreakage was about 30%.

The bonus varied from 20c. to 40c. per ft. over the base rate, depending on the class of material encountered. At the start it was estimated that a reasonable progress would be 8 ft. per day with three 8-hr. shifts. A bonus system based on this advance gave 40c. per ft. in excess of the 8 ft. Thus it was expected that the monthly advance would not fall below 248 ft.; as a matter of fact, 10.8 ft. per day was averaged, giving a monthly progress of 330 ft. The standard tunnel crew consisted of 16 men in untimbered tunnels and 23 men in timbered tunnels. In dry tunnels miners received $3 per day, and muckers and timbermen $2.50 per day; in wet tunnels each man received 50c. more per day. The itemized account is given in Fig. 7.

TABLE 7. DAILY COSTS, LOS ANGELES AQUEDUCT TUNNELS

Average daily progress	10.8 ft.
Average cost, direct charge	$51 10
Equipment	6 50
Pipe lines and operation	.46
Roads, trails and maintenance	1.40
Buildings	1.49
Surveys	.24
Division administration	2.12
Executive administration	1.92
Miscellaneous losses, test, phone, operation, etc.	.53
Grand total cost per foot of tunnel	$65 76

Springfield Water-Works--I

SYNOPSIS—Organization of the Springfield, Mass., water-works; plant and equipment, which include a thoroughly uptodate shop, garage and stable and a material yard equipped with electric-locomotive crane. A later article will describe other features of interest.

The public water-supply of Springfield, Mass., dates from 1843. In 1872 the water-works was purchased by the city from the private company that had owned and operated it up to that time. There are at present two principal sources of supply—filtered water from the Westfield Little River and water from impounding reservoirs in the adjoining towns of Ludlow and Belchertown. The total population served is estimated to be 101,735, not including the town of Ludlow, which also uses water from the Ludlow reservoirs. The number of metered taps is 13,191 or 95.5 per cent. of all services.

The water-works of Springfield, Mass., is managed by a board of three water commissioners, of whom the mayor, ex officio, is one. Under this board are three separate divisions—one under a superintendent of water-works, one under a water registrar and clerk of the board, and one under a chief engineer. The duties of these officers are apparent from their titles. This article is confined to the division under the superintendent of water-works.

The superintendent, Alfred E. Martin, has three principal assistants—an assistant superintendent, who has general supervision of installing mains, two construction gangs, a gate-inspection service gang, service-pipe gangs, and of emergency work; a master mechanic of shops and plant; and a superintendent's assistant, who has general oversight of many small details. The organization scheme is shown in the chart, Fig. 1.

The plant proper consists of a shop, a stable and a materials yard (Fig. 2). The shop was completed in 1913 at a cost of $50,000 and embodies what were then considered the best features of a number of similar plants inspected and studied by the Springfield officials. The brick building is 50x140 ft., of which the shop takes up one-half (see Fig. 2). The basement of the whole has a concrete floor, walls, columns and ceiling, and is used for a storeroom, boiler room and locker room. There are two shower baths in the basement for the use of the day laborers and one for shop employees on the second floor. Above the basement the building is of slow-burning mill construction, except the garage and stable, which have reinforced-concrete floors, walls and columns. The building has an elevator and electric power and is fitted throughout with reels of hose for fire protection and self-closing fire-doors.

The first floor of the shop is finished off for offices and as a storeroom for small stock. The second floor

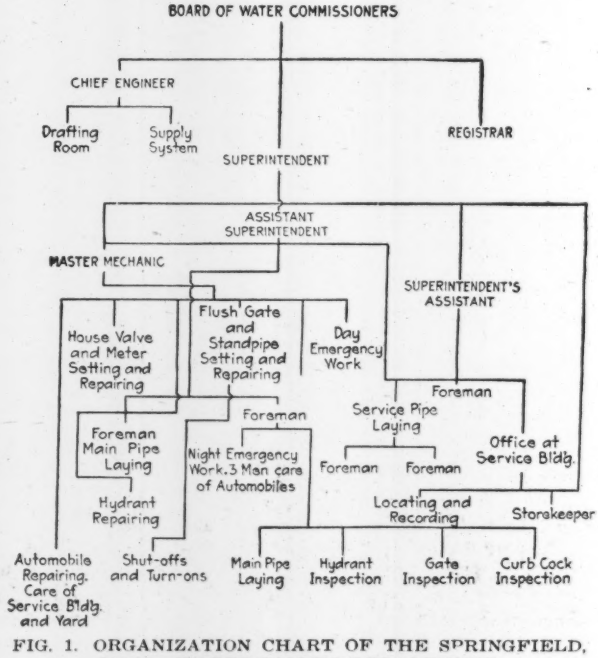


FIG. 1. ORGANIZATION CHART OF THE SPRINGFIELD, MASS., WATER-WORKS DEPARTMENT

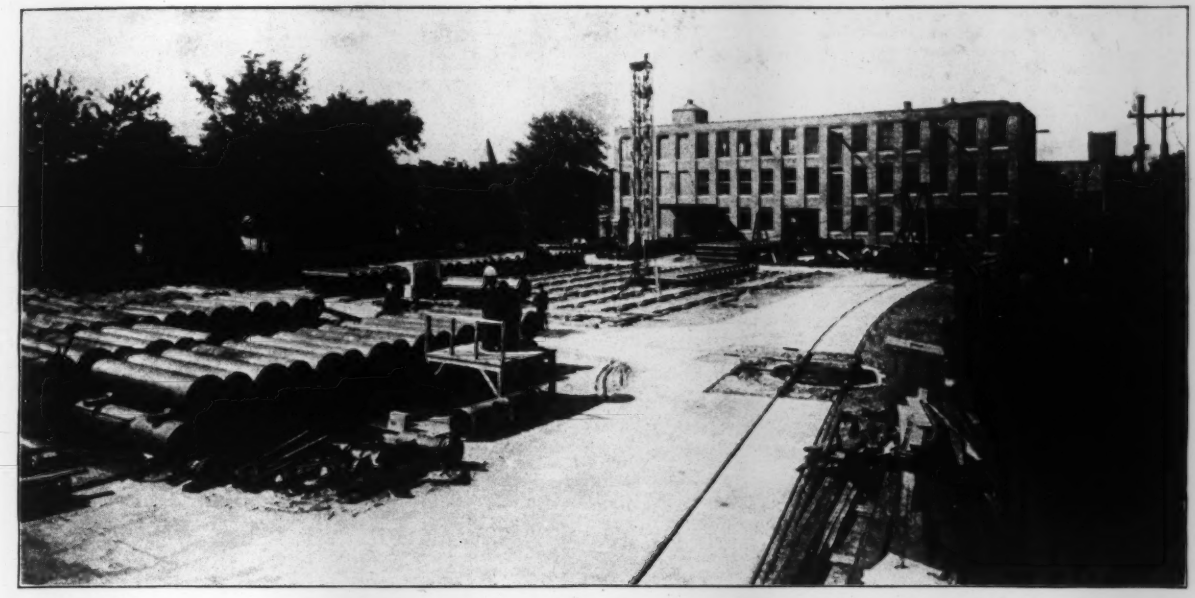


FIG. 2. GENERAL VIEW OF SHOP, GARAGE AND STABLE AND YARD, SPRINGFIELD, MASS., WATER-WORKS

has a meter-repair shop (Fig. 3) and a meter-testing room (Fig. 4). The former is equipped with a number of machine tools, such as a lathe, a 6-in. pipe-threading machine, a small pipe-threading machine, a drill press, an air compressor and various small tools. Meters are cleaned by sand blast, repainted and sent out as bright as new.

The meter-testing room is equipped with standard meter-testing apparatus (in front of windows, Fig. 4) consisting of a Gow meter-clamping device, with a Mueller & Chapman quick-opening valve on a 2½-in. diameter delivery main, for testing meters up to 2-in. diameter services. A 36-cu.ft. weighing tank is used, and all meters are tested for flows of 10 and 100 cu.ft. For large meters there is a 6-in. main connection (right of Fig. 4) with a slip-joint clamping device and a Chapman quick-operating valve. The large measuring tank holds 330 cu.ft. and has a calibrated glass gage. A ½-ton

FIG. 3. MACHINE SHOP OF SPRINGFIELD, MASS., WATER-WORKS PLANT

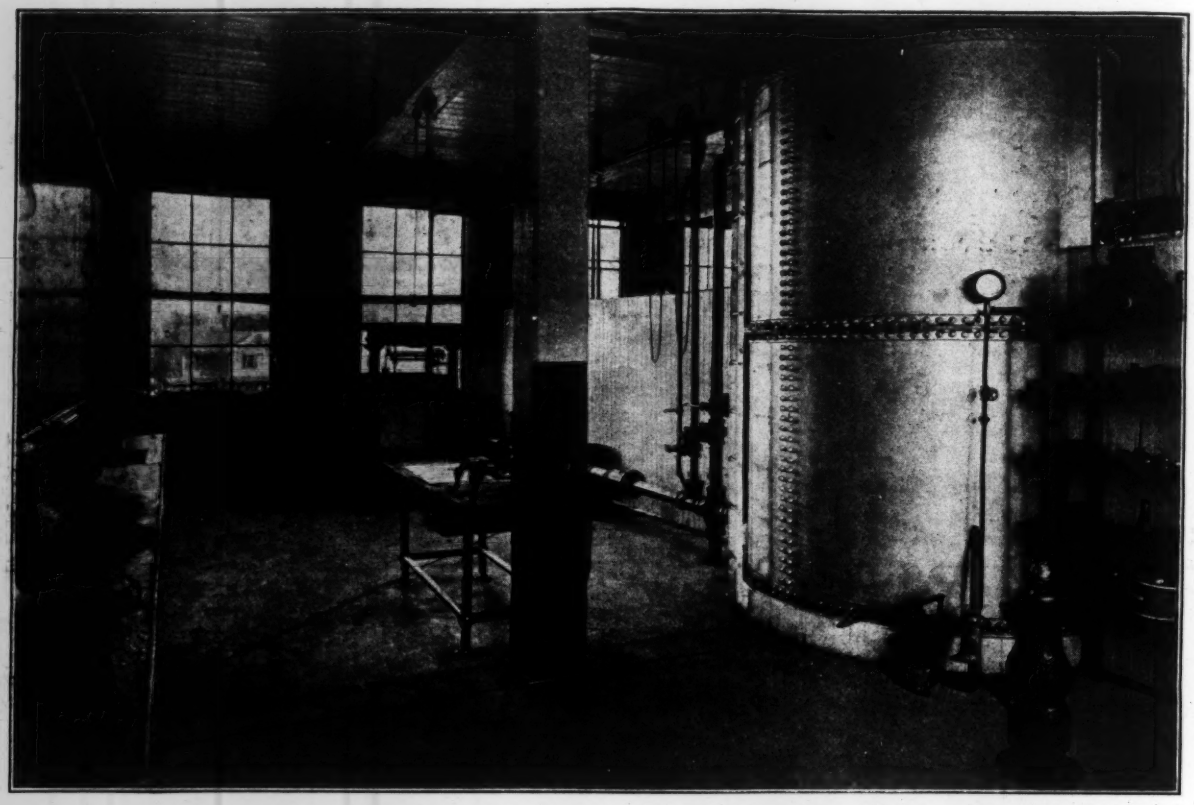


FIG. 4. METER-TESTING ROOM OF SPRINGFIELD, MASS., WATER-WORKS PLANT

chain is used to place large meters in position to test. All meters are tested one at a time.

A small electric-driven pump and pressure tank make possible a pressure of 400 lb. for testing valves (foreground of Fig. 4) and also the testing of service cocks. Regulation corporation cocks are tested to a pressure of 220 lb. per sq.in. The equipment of the shops also includes three Smith pipe-cutting machines for making taps in mains up to 12 in. in diameter; when it is necessary to make larger taps a machine is hired.

The third floor is a storeroom and at present a museum of water-works relics, including some interesting pieces of pipes, valves and fittings of real historic interest. The most curious is a section of glass pipe about 2 in. in diameter, incased in cement mortar, dug up a few years ago in the vicinity of the United States Arsenal.

The first floor of the right-hand end of the building (Fig. 2) is occupied, as noted, by a garage and wagon shed, separated from the shop by a concrete wall. There are eight water-works automobiles, a 3-ton truck, a 2-ton truck, an emergency car, a repair car for making shutoffs, etc., a superintendent's car, an assistant superintendent's car, one for the use of the meter foreman and one for the chief engineer of the water-supply. The stable is on the second floor; horses are kept for four wagons. There are harness rooms, a steam-heated blanket dryer, a washstand and other facilities of a first-class stable. The third-floor storeroom extends over the stable, with a partition wall between.

The yard is small for the amount of material handled, but has unusual facilities for storing and handling. There is an electric locomotive crane capable of lifting 1½ tons on a 50-ft. radius. The standard-gage track consists of one main line (left of Fig. 6) which is directly connected with the New York, New Haven & Hartford R.R. This track extends the length of the yard, about 10 ft. from one side, with two branches at right angles extending the width of the yard about 100 ft. apart. Iron turntables are used to transfer the crane from the main to the branch tracks, enabling it to cover practically every foot of the yard. Power for the crane is transmitted from a central tower through a flexible cable, of which the slack is taken up on a self-winding drum.

In order to have sufficient storage the pipes are piled in tiers, sometimes 12 or 15 pipes high. To facilitate handling the lower ones the bottom tier is laid

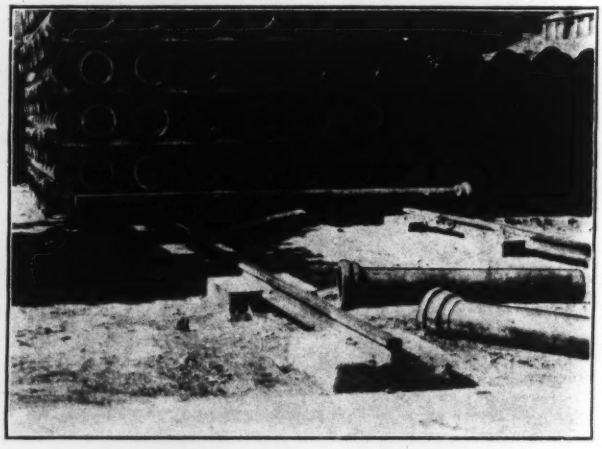


FIG. 5. RAILS FOR PILING TIERS OF PIPE

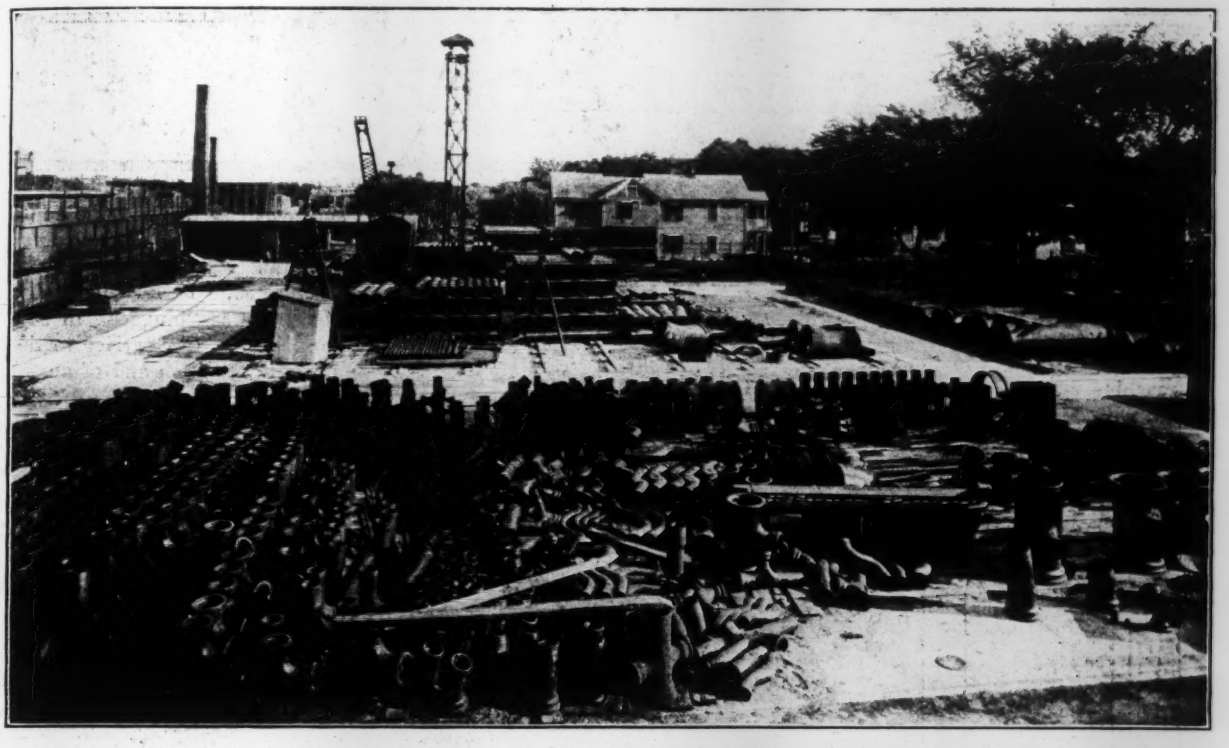


FIG. 6. MATERIALS YARD OF THE SPRINGFIELD, MASS., WATER-WORKS PLANT

on two parallel railway rails molded into concrete blocks, as shown in Fig. 5. There are facilities for painting the pipes and special castings, including a small hoist and dipping tank for the special gate boxes. In the far corner of the yard are sheds for storage and for shelter of workmen employed on various jobs about the yard.

The whole force employed in the shops and yard is about 15 men. The emergency force consists of any of the men under the shop foreman during the day. There are three men constantly available at the garage until 1 a.m., after which there are two men until the shop opens at 7 a.m. There is a private telephone between the garage and the foreman's house, which is near-by.

Missouri River Improvement Work May Be Stopped

The river transportation enthusiasts in and around Kansas City are considerably perturbed because Lieut.-Col. Herbert Deakyne, Corps of Engineers, U. S. A., the engineer officer in charge of the Missouri River district, has recommended to his superior that the improvement of the Missouri River be restricted to snagging operations. Mass meetings of those interested are being held in protest against these recommendations and the usual local press campaign in favor of the river improvement is in full swing. The gist of Col. Deakyne's report is as follows:

The commerce to be considered is present and prospective, and the advantages that commerce will obtain from the improvement are the actual saving in freight charges by the use of the river and the effect on railroad freight rates produced by the possibility of water transportation.

The items that are not benefited by improvement of the river are the sand and gravel barged and the logs and lumber rafted. They amount to 309,684 tons. Ferry traffic is not counted in reporting the commerce of the Missouri River.

Eliminating the logs and the sand and gravel barged, the remaining traffic is 37,551 tons, divided into the classes which are carried over average distances of from 8 to 291 mi. The total freight charge on this traffic was about $41,000. The Kansas City Missouri River Navigation Co., the only through line on the river, operating between Kansas City and St. Louis, charges 80 per cent. of the railroad freight rates. Assuming this to be the relation between the rail and water rates for the entire traffic, the saving to shippers by the use of the river in 1913 was about $10,000. It is evident that this saving is entirely inadequate to warrant the serious consideration of an expenditure by the Government of $1,100,000 per year in interest and maintenance.

In some cases the improvement of a stream is justified even at high expense, where the absence of rail transportation leaves the territory along the stream entirely dependent on water transportation. This is not the case with the Missouri River between Kansas City and the mouth. Over a great part of the distance the river is paralleled with railroads on both banks, and there are few localities that have no railroad within easy reach.

There remains to be considered the question of prospective commerce, and in my opinion the improvement must stand or fall by this criterion. In the survey report upon which the present project is based, the prospective commerce was given by the president of the Missouri Valley River Improvement Association as 1,000,000 tons. That estimate was based upon a 10-ft. channel with six or seven boats especially adapted to the navigation of the Missouri River, handling cargoes of 2000 tons each, and making one round trip per week during a navigation season of ten months. The fact is that anyone can make almost any estimate he sees fit as to the prespective commerce. I am of the opinion that the present and reasonably prospective commerce on the Missouri River between Kansas City and the mouth is not sufficient to warrant the continuance of the present project. I think the snagging should be continued, so that commerce may have the advantage of the natural depth of the river and the river shall not be allowed to become entirely obstructed. Any attempt at partial improvement seems futile, because the draft of boats, and therefore their cargoes will be determined by the least existing depth. If the river is improved at all, the work should be complete as to protection of banks and control of channel. For the same reason it seems to me injudicious to spend any money in keeping up the works already built. If in another generation the improvement of the river becomes advisable, the present works will be of little value and in the meantime the channel depths in the portions that have been improved are not likely to be less than in the portions wholly unimproved.

Field and Office

Reinforcing Concrete Columns to Carry Additional Stories

By Paul R. Prufert*

It was necessary to resort to a somewhat unusual method of construction in the addition of two stories on the machine shop of the Washburn-Crosby Co. mill in Minneapolis, Minn. This structure was one of the first

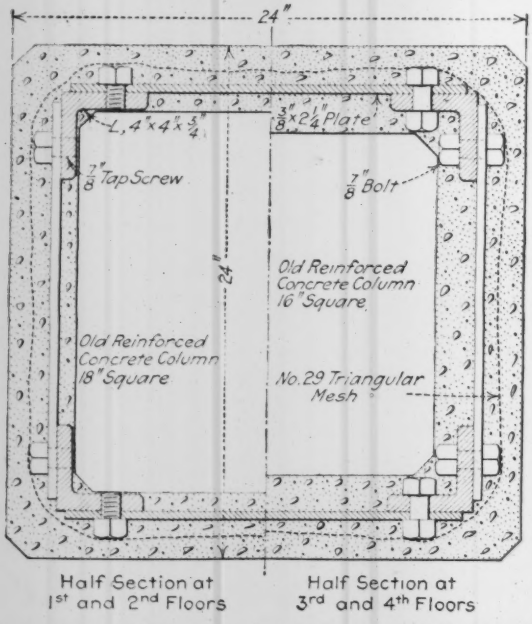


FIG. 1. HALF SECTIONS THROUGH COLUMNS, SHOWING METHOD OF REINFORCING

reinforced-concrete buildings erected in that city and covers an area of 60x78 ft. It is five stories high, has brick bearing-walls strengthened by brick pilasters and six interior reinforced-concrete columns. The floor construction is of the girder type, 6-in. slabs with two-way reinforcement. The wall footings went down to bedrock, about 9 ft. below grade, but no accurate data could be found regarding the size or depth of the six column footings. On figuring the old structure, the writer found that the concrete columns were not of sufficient strength to carry any additional stories; they were actually too weak for their present requirements. Neither was the roof-slab, of course, strong enough to be utilized as a floor. In fact, the only parts of the building that would stand additional loading were the walls.

Another condition that added to the difficulty was that neither the continuous use of the elevator nor the lower four floors of the building could be interfered with, and as two floors were used for flour storage, it was essential to guard against leakage. Obviously the first step was to examine the column footings, to see whether they rested on bedrock and were of sufficient size to carry the additional load. They were uncovered, and five of them were found to go down to the limestone ledge, while the sixth one rested on a layer of shale about 18 in. thick. To secure solid bearing this footing was "plugged" in sections of about 1-ft. width and in that way carried to the ledge. All of them were in good condition and of ample size.

As the old concrete columns could not be utilized, it was decided to transmit the load of the new stories to the footings by means of structural-steel members built around the old columns. Four holes were cut in the floor-slabs at each column and 4x4x¾-in. angles lowered down from the roof to the footings. They rested on a structural-steel base, made in two parts and bolted together. The angles were connected by ⅜x2¼-in. straps every 6 in. alternating, ⅞-in. tapscrews being used for connections at the two lower floors, where the original

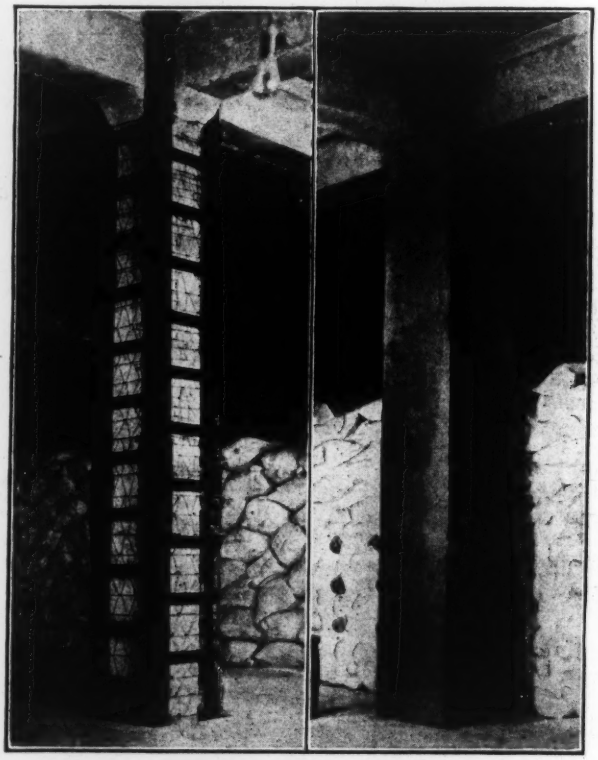


FIG. 2. REINFORCED COLUMNS BEFORE AND AFTER BEING FIREPROOFED

columns were 18 in. square, and bolts on the upper two floors, where the old columns were 2 in. smaller. After the angles were connected up, No. 9 triangular mesh was wrapped around them and the concrete for fire-protection poured, one floor at a time. The finished columns were 2 ft. square, an increase of 6 in. over the old ones.

The old roof-slab and the fifth-story columns were then wrecked, structural-steel caps bolted to the tops of the

*Engineer, John Wunder Co., building contractors, Minneapolis, Minn.

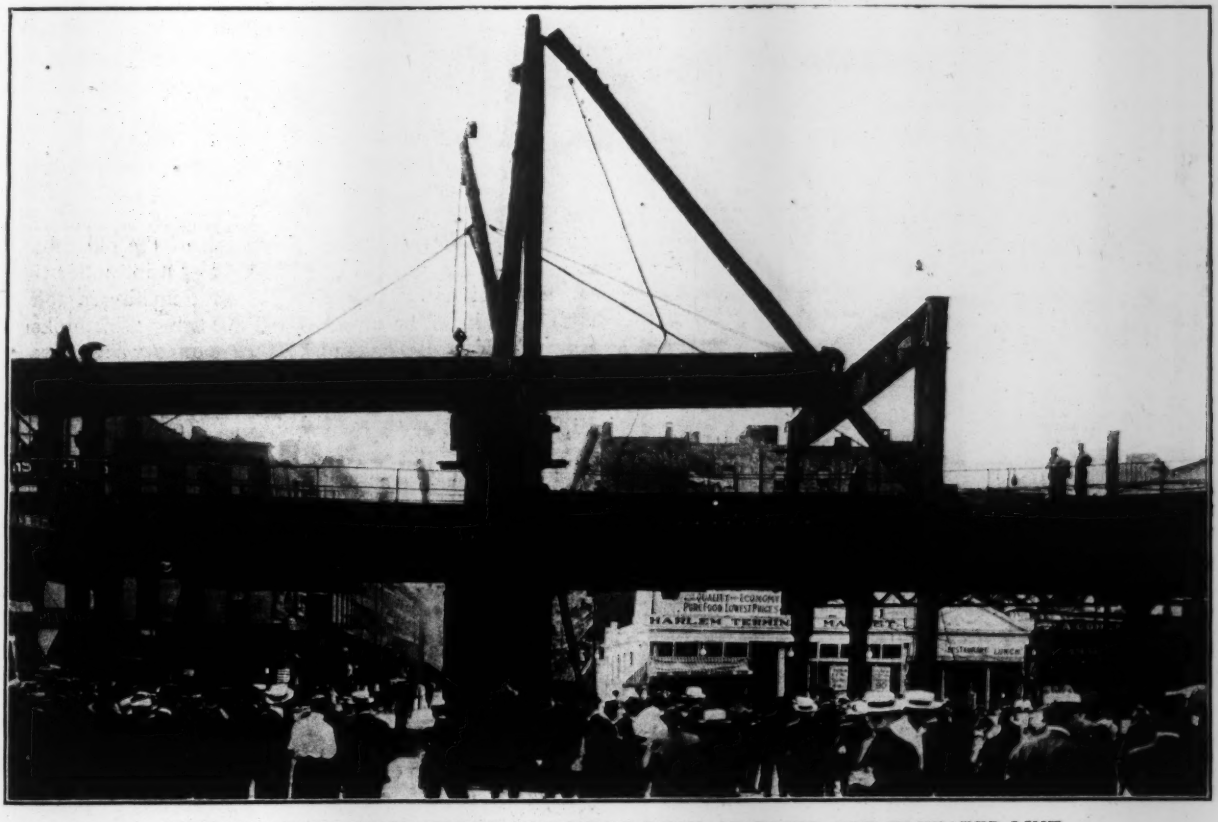
angles and the columns for the new superstructure poured on these.

The new floors were of flat-slab construction, without drops or column heads. Three weeks after the old roof-slab was wrecked, the new one was poured, and not a single sack of flour had been spoiled although the rainfall had been considerable during that period and stopped construction for several days.

The work was done by John Wunder Co., Minneapolis, Minn., contractors and engineers, with the writer in charge of design and construction.

Handling Heavy Girders on Elevated Railway

Double-decking the Third Ave. Elevated Line where it is to cross the Harlem River, New York City, at 129th St., on the new double-deck bridge that is being built to take the place of the old single-deck structure involves the handling of girders that are longer and heavier than those used at most other points on the third-tracking work. The accompanying view shows a stiff-leg derrick set up in a rather awkward location, handling a 17-ton girder of 88-ft. span. One of the stiff-legs rests on the elevated structure, running lontitudinally with it. The other stiff-leg juts out over the street at right angles to the first leg and is carried on a timber bent resting on the street. The derrick is at work on the elevated structure at 128th St., erecting the express deck. Where possible in third-tracking the elevated railways of New York a derrick car was employed, but this car was designed for handling loads only up to eight tons and could therefore not be used on the heavier work on Third Ave. from 127th to 129th St. The car was described by J. M. Ryan, assistant engineer, Snare & Triest Co., in *Engineering News*, of May 20, p. 986.



SETTING A LONG-SPAN GIRDER IN DOUBLE DECK OF THIRD AVE. ELEVATED LINE
Photograph obtained from Broderick & Bascom, which furnished ⅝-in. hoisting cable of derrick

Reconstruction of Street-Car Tracks at Kansas City, Mo.

By E. B. Murray*

With the repaving of the main streets in the downtown district of Kansas City, the Metropolitan Street Ry. Co. took the opportunity to reconstruct its tracks and build a new foundation in advance of the paving work.

In each street it was necessary to keep the car lines open. As various routes converge on these streets, the traffic is very heavy, and during the busy hours there were 134 cars per hour to be handled. The street was closed to ordinary vehicular traffic, however, for about four blocks at a time. A temporary track was laid for the street cars at each side of the street, close to the curb line, so that passengers could be landed on the sidewalk. Plank crossings for pedestrians and street vehicles were placed at street intersections, and extra trolley wires over the temporary tracks were attached to the original span wires.

It was decided to reconstruct both tracks at once and to close the street entirely to ordinary vehicles, rather

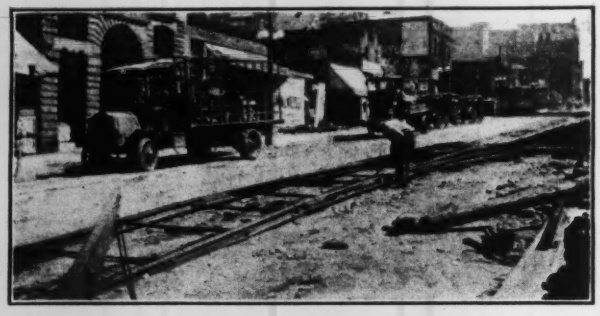
*Consulting Engineer, 920 Walnut St., Kansas City, Mo.

than to work on one track and keep one side of the street open. In the latter case it would still be necessary to use the open side of the street to store material, so that it would not be possible to leave much room for traffic. Consequently both the construction work and the street traffic would be badly hampered. Work was done on only one street at a time, and in sections four blocks in length. The work on a section occupied from 18 to 21 days, ten days being allowed for the setting of the concrete base.

Portable Turnouts to Temporary Tracks

The temporary tracks were connected to the permanent tracks by turnout curves of 100-ft. radius and Kerwin portable turnouts or crossovers. These turnouts are placed directly on the rail, an incline or feather rail engaging and raising the car wheels. This is shown in the accompanying view. In this way the crossover can be used without any disturbance of the track or paving.

Where the temporary tracks intersected the tracks from a side street, the same idea was utilized in a portable crossing. This was made similar to a regular crossing, but was laid directly upon the rails and had feather rails projecting in four directions so as to allow the cars coming in any direction to mount the crossing and pass



PORTABLE TURNOUT LAID ON STREET-RAILWAY TRACK TO DIVERT CARS TO A TEMPORARY TRACK

over. It is believed this is the first time the principle has been applied to a crossing.

The temporary tracks were laid with second-hand ties and T-rails of about 50 lb. per yd. The rails were not reinforced by tie-rods or rail braces, but were simply spiked to the ties. The tracks were ballasted with 6 to 8 in. of material excavated from between the tracks (before traffic was diverted). This consisted of earth and broken stone and was laid upon the street paving. The rain water was taken care of by leaving a narrow ditch between the curb and the end of the tie, the ballast not being filled close against the curb.

As soon as traffic was diverted to the temporary tracks the old paving and tracks were torn up, the material being removed by the railway company's work cars. Then the concrete base was broken up to allow of removing the old ties. This was done by hand sledging, the concrete being split into large pieces by means of chisel and sledge.

Constructing New Track Foundation

The excavation for the new subgrade was done by hand, and was carried down to 6 in. below the bottom of the ties and to the top of the old cable railway foundation. It was decided not to disturb this old foundation, as it was approximately 4 ft. thick and very solid. The old concrete that was broken up was shoveled into cars and used for ballast and filling on open-track lines. The new foundation was made of fresh material. It was placed after the new ties had been put in position, blocked up to the required height and the rails securely spiked to them.

The material for the new concrete base was brought to the point of distribution on work cars or trains and deposited adjacent to the mixer in the middle of the street. These cars are constructed of steel and each carries three self-dumping pockets of 6-yd. capacity. The mixing plant was mounted on a small push car and was motor driven. Its loading platform was so constructed as to allow the men to load their wheelbarrows at the side of the street and empty them directly into the hopper of the mixer.

For the distribution of the concrete there was a special end-dump car, having a capacity of one batch, or approximately ⅓ cu.yd. It had originally a hinged end, but it was found impossible to get the joints clean after dumping and to make the joint tight enough to retain the grout. The car was then redesigned so as to have the entire body dump forward. This car ran on the new track. Wherever it was possible, the mixer emptied directly into chutes conveying the concrete to the point of distribution.

The new concrete foundation on top of the old cable-track foundation was placed to a depth of 6 in. under the new ties and half way up on the rail, making a total depth of 21 in. The mix was 1:3:6, using 2-in. broken stone and Kaw River sand. The concrete was allowed to set for ten days. Then the railway company laid granite blocks on both sides of the rails and wood blocks (with asphalt filler) between the tracks.

Relaying Track

The new tracks are laid with white oak ties 6x8 in., 8 ft. long, 36 in. c. to c. These are entirely embedded in the concrete, and are estimated to have a life of 20 to 30 yr. The rails are secured by ¾-in. screw spikes. The spike holes were bored by electrically operated augers, and the same machines, fitted with socket wrenches, were used to screw in the spikes. Current was taken from the trolley wires.

The original rails were relaid in this new construction. As soon as the old rails had been exposed and the splice plates removed, the dirt and scale around the joints were removed by a portable sand blasting plant. This consisted of a motor-driven compressor (50 cu.ft. of free air per minute) and a sand tank, mounted on a push car and having a 50-ft. hose line with nozzle and valve connections. Current was taken from the trolley line. In this way a joint could be cleaned in about 10 min. The joints on the relaid rail have plates held in place by bolts and tie-rods.

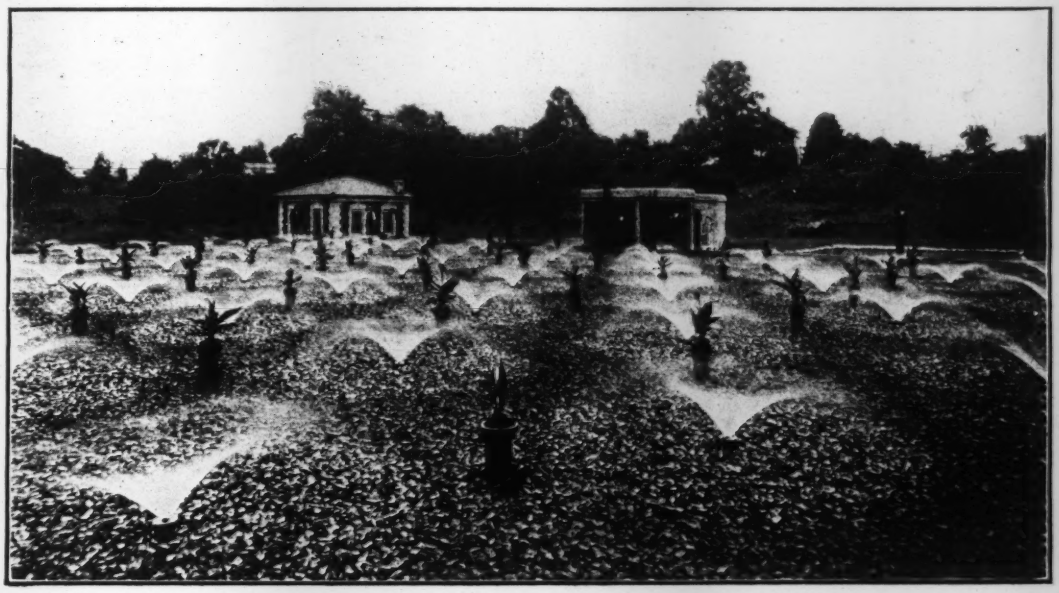
Current for the augers, spike wrenches, sand-blast machine and concrete mixer was obtained from the trolley wire over the permanent track by means of an extension pole with hook on the end. This did not interfere with the cars, as these were operated from temporary wires placed over the temporary tracks.

The Metropolitan Street Ry. Co. carried on the reconstruction with its own forces, the number of men varying according to the different stages of the work.

At some points where track intersections were involved the work was done at night to avoid interruptions to traffic. The work was carried on with a view to rapidity and to the convenience of adjacent business houses, the results being satisfactory to all parties. The method of handling the work, as well as much of the equipment used, was devised by A. E. Harvey, Chief Engineer of the company, who was in direct charge of the work.

Ornamental Plants on Sewage Bed, Harrison, N. Y.

The sprinkling filters of the sewage-works of Harrison, N. Y., have been ornamented with plantings at the intersections of the sprays. The disposal works, even when the sprays are not in action, is sightly, but when the sprays are in operation the effect is very attractive, and the expense was trifling.



ORNAMENTAL PLANTS ON SEWAGE SPRINKLING FILTER, HARRISON, N. Y.

As will be seen from the view, 2-ft. lengths of 6-in. vitrified pipe, filled with earth, are embedded in the broken stone of the filter, and the moisture from the sprays and in the stone produces thriving plants.

The photograph also shows that in designing the pumping station a slightly greater expense than was required by the stern necessities of the case was incurred by adding a little cut-stone trimming.

The effect both of the plants and of the stone trimmings on the general appearance of works devoted to such a purpose as sewage disposal is worth while, and engineers should more frequently insist upon such slight additional expenditure.—Alexander Potter, Consulting Engineer, New York City.

Hydrated Lime to the amount of 0.1 cu.ft. to each 1 cu.ft. of cement is to be used for all concrete in the culverts, bridges and pavement of the 6½-mi. section of the Coleman du Pont Road in Sussex County, Delaware. A concrete pavement is now being constructed in Somerset County, Maryland, under the direction of Henry G. Shirley, Chief Engineer of the State Roads Commission, in which a percentage of hydrated lime is also used. Previous experience in Maryland with hydrated lime for pavements was noted in "Engineering News," Mar. 11, 1915, and June 3, 1915.

Laying Submerged Outfall Sewer in Surf

By A. J. Cleary*

To lay cast-iron pipe in calm water does not present any unusual engineering difficulties. To perform the same work through a heavy surf, under such adverse conditions as prevail at Baker's Beach, near the entrance to San Francisco harbor, demands ingenuity.

The Baker's Beach outfall is an 18-in. cast-iron, flexible joint sewer pipe, which extends 800 ft. from the shore line and will have a discharge chamber 32 ft. below mean low tide. On account of the difficulties incident to laying the submerged pipe at the location specified, the city engineer allowed the contractor to try several methods, only insisting that the finished line and outfall absolutely conform to the specifications. The two unsuccessful and the final and successful method tried are here described.

First, the whole 800 ft. was laid just above high-tide mark parallel to the water's edge, the joints leaded and the two ends sealed and buoys attached. An attempt was then made to float the pipe into position at right angles to the shore, remove the buoys and allow the pipe to

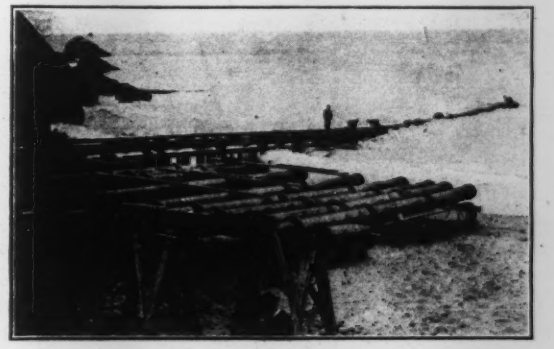


FIG. 1. SECOND ATTEMPT TO LAY OUTFALL SEWER

*Assistant City Engineer, San Francisco, Calif.

settle to its final position. This trial resulted in failure, as described in *Engineering News* of Feb. 18, 1915, page 325.

It was then decided to try laying the pipe in short sections. The lead was melted out of every fourth joint, producing sections 48 ft. long, two of which were subsequently united. A skidway 24 in. wide was built from

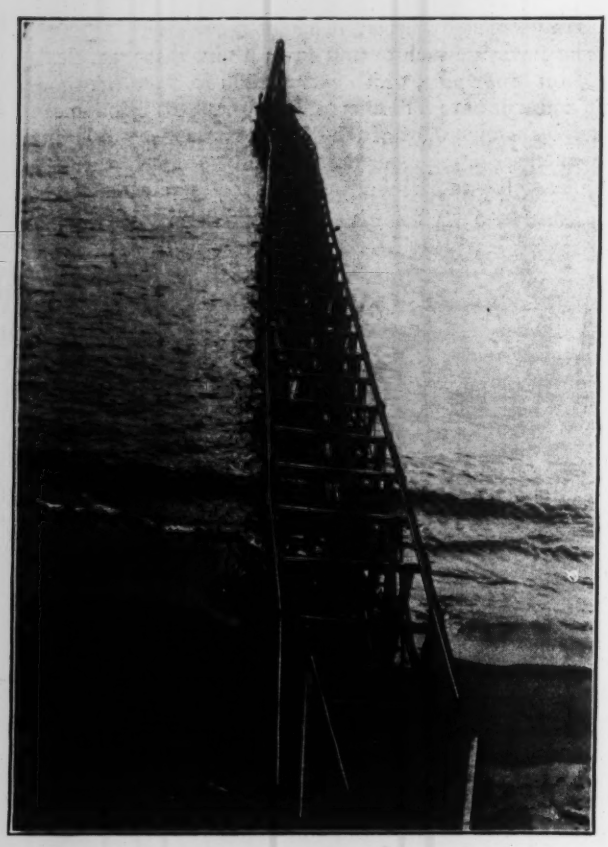


FIG. 2. OUTFALL SEWER SUCCESSFULLY LAID FROM TRESTLE, SAN FRANCISCO

shore to beyond the first line of breakers and a platform consisting of five 8x8-in. runways was erected adjoining the skidway, as shown by Fig. 1. The 48-ft. sections of pipe were placed on this platform so that they could be rolled in rapid succession onto dollies that would carry them down the skids. The three joints in each 48-ft. section were recalked while the pipe was on the platform. A barge was anchored 800 ft. from shore, and a hand-winch set up on the barge. The intention was to pull a 48-ft. section far enough out on the skidway to allow the next section to be rolled in line behind it. The joint was then rapidly poured and calked and four sealed empty barrels wired to each pipe length to buoy up the line, as in the first attempt. When the two 48-ft. sections were joined and the barrels attached, the winch on the barge was operated and the pipe pulled 48 ft. seaward. The above operation was then repeated. Anchors, all equipped with block and tackle, were placed in two lines, each line 50 ft. from the proposed center line of the pipe. As the pipe was pulled seaward, lines from the anchors were fastened to it and kept taut to hold it in its proper position.

June 5 was chosen as a suitable day for launching the pipe in the manner just described, as all indications on the day before were favorable for a calm sea and little wind. Accordingly, work was begun on the morning of that date and all went successfully until 240 ft. of pipe had been floated to sea. With the turning of the tide a strong breeze set in through Golden Gate and heavy swells began to break on the beach, greatly impeding the work. Finally, a particularly high breaker swept the pipe off the skids, stove in many of the barrels and the half-laid line sank gradually to the bottom of the ocean.

It was then decided that the surest and safest method of laying the pipe would be to drive a pile trestle along the proposed line of the sewer, lay the pipe thereon and later lower it into place. Forthwith a pile trestle as shown in Fig. 2 was built and the pipe laid and successfully lowered. The reason that this was not done originally was because during the winter and spring seasons the ocean in the vicinity is not calm enough for periods sufficiently long to permit of anchoring a pile driver along the line of the sewer.

Testing Surveyors' Tapes by the Canadian Government

By J. A. Macdonald*

The Surveyor General's office of Canada is required by law to test and certify to the length of Dominion surveyors' standard tapes. To carry out this work and make tests of other surveying instruments a special building has been erected and equipped.

This building consists of two rooms, an office with vestibule and the testing room. The testing room is 150 ft. long by 10½ ft. wide. The 4-ft. walls consist of five thicknesses of brick, a 1-in. air space, sheeting, tar paper, 18 in. of shavings, sheeting, a 4-in. air space, tar paper and double sheeting. Between the ceiling and the roof is a layer of shavings 4 ft. thick and beneath the floor a 2-ft. layer of shavings, under which is 5 ft. of cinders to solid rock.

The only openings other than the entrance are two air intakes, one at each end of the room, and four adjustable ventilators in the ceiling. The air intakes are at the base of the wall and the quantity of air admitted is easily regulated or completely shut off. Electric fans at each end of the room give circulation to the air and insure a uniform temperature throughout the room. The exit into the outer room consists of two doors, with a 4-ft. air space between. The outer door is of the ordinary type, but the inner one is an insulated, air-tight door of the kind used for refrigerators.

To maintain a temperature of 62 deg. F. in cold weather there is an electric heating system. The apparatus is of the three-heat type, giving low, medium and high heat. The degree of heat is controlled from a marble switchboard. The heater and wire conduits are shown near the base of the wall in Figs. 1 and 2.

Tapes are compared with a base, the ends of which are lines scratched on polished surfaces. The distance between the marks is determined by an invar bar, 4 m. long, standardized at the International Bureau of Weights and Measures at Paris. The marks are established on

*Topographic Surveys Branch of the Department of the Interior, Ottawa, Ont.

concrete piers extending to bed rock. The marks are at intervals of 0, 4, 8, 10 and 12 m., at 50 ft., at 16 and 20 m., 66 ft., 24, 29 and 30 m., 100 ft. and 32 m.

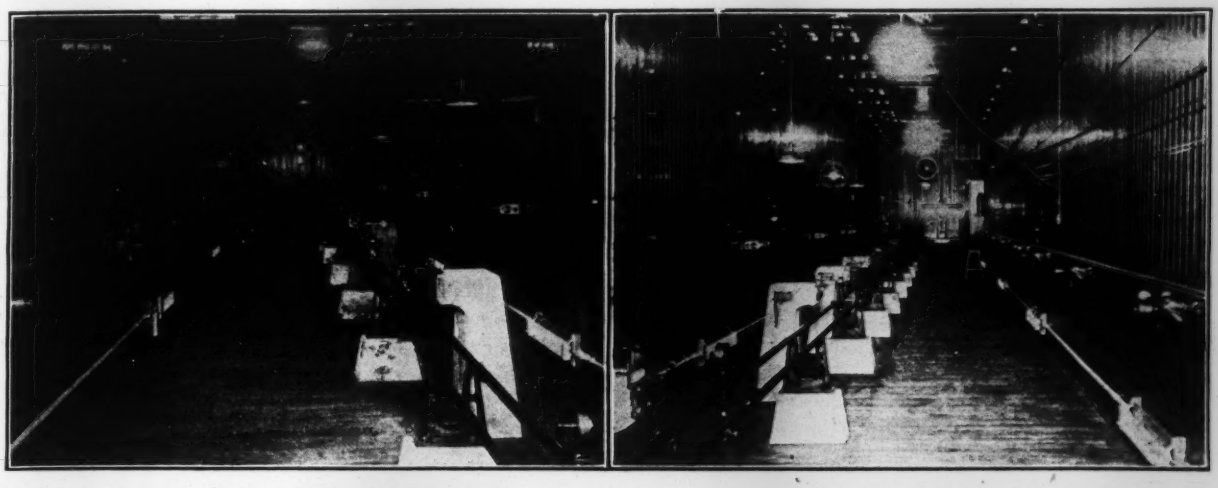
The invar bar is kept in a wooden case and only the graduations in use are exposed. The temperature of the bar is taken from two thermometers inside the case, read through openings in it. There are 12 thermometers suspended in protective guards at equal intervals along the walls of the room. These are graduated to 0.1 deg. C. and can be estimated to 0.01 deg. C. A mean of the readings gives an accurate determination of the room temperature.

The bar is supported on a carriage traveling on rails and is quickly and easily moved to any part of the base (Figs. 3 and 4). The carriage with microscope has slow-motion adjustment for both vertical and horizontal movements. Small 2-cp. electric lamps to illuminate the graduations are mounted on the concrete piers.

The track which carries the bar trucks is supported every 2 m. by cast-iron U-shaped frames, resting on concrete piers carried down to solid rock. The U-shaped frames permit the tape when being tested to sag unobstructed. The pulley supports (see Figs. 1 and 2) for suspending a tape may be placed at any point along the track and clamped in position by two thumb-screws.

The cord or wire from the end of the tape passes over a grooved pulley and is attached to a weight to give the standard tension desired. The pulley has ball bearings and may be aligned vertically and horizontally by means of thumbscrews. Intermediate ball-bearing pulley supports may be placed as desired.

The testing room is also equipped with the ordinary apparatus for testing transits for the stadia constant and for testing transits and levels for errors of collimation. All surveying instruments sent out to Dominion land surveyors, as well as those used in the government geodetic surveys are tested here before shipments. The building and equipment are in charge of W. C. Waye, of the Topographic Survey staff, from whom the writer obtained the foregoing information.



FIGS. 1 AND 2. VIEWS OF THE TAPE-TESTING ROOM OF THE TOPOGRAPHIC SURVEYS BRANCH OF THE DEPARTMENT OF THE INTERIOR, OTTAWA, ONT. (FROM OPPOSITE ENDS OF THE ROOM)

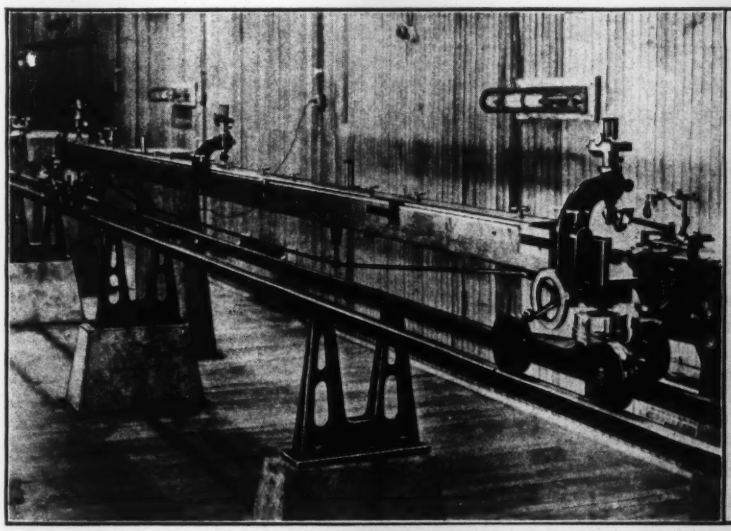


FIG. 3. FOUR-METER INVAR BAR FOR DETERMINING LENGTH OF STANDARD BASE

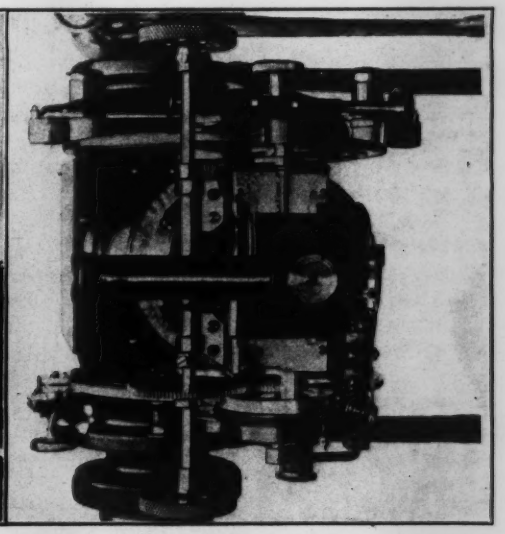
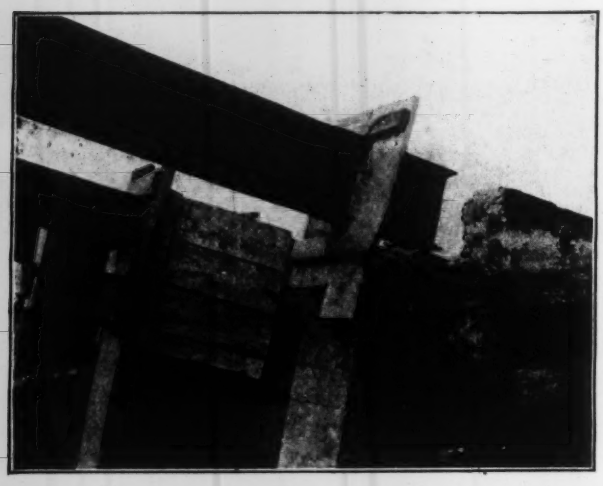


FIG. 4. NEAR VIEW OF MICROSCOPIC COMPARATOR, INVAR STANDARD

NOTES

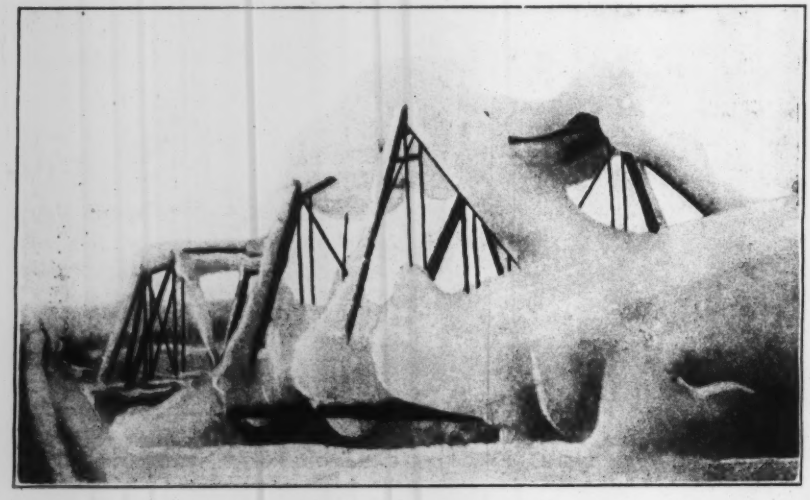
The Steel and Concrete Floor Supports of the Sheepshead Bay automobile racetrack were joined as shown in the accompanying illustration. The track is a 2-mi. oval, 70 ft. wide, with a wood floor made of 2x4-in. timbers on edge. The floor is supported on concrete ribs, or walls, spaced 6 ft. 7 in. on the straightway sections and 6 ft. 8 in. (on the outside



SHEEPSHEAD BAY MOTOR RACETRACK FLOOR CONSTRUCTION

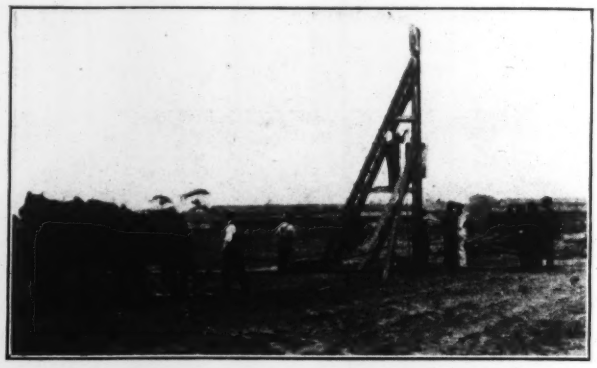
edge) on the turns. The concrete is carried to a height of 6 ft., above which the floor is carried on 10-in. I-beams. The concrete walls were constructed with the corner omitted, as shown, and the base for the I-beam was left about 2 in. below grade. The I-beam was then blocked up and inclosed in special forms, as shown. A ¾-in. bolt was placed crosswise through the web for an anchor.

Cutting Lock Jointed Steel Sheetpiling, where the thickness of each joint was 2¼ in., was easily and quickly accomplished recently by an oxyacetylene torch on a bridge-foundation job in Florida. About 860 ft. of piling had to be cut off to a uniform height. The work was in connection with the construction of the piers for a large drawbridge over the ' . John's River at Jacksonville, Fla. The piling was the Lackawanna Steel Co.'s lock-joint type with ⅜-in. web. On account of the thickness to be cut at the lock joints the operator was handicapped in making speed, but nevertheless between 40 and 50 ft. of cutting was done in 7 hr., with only one torch. Prest-O-Lite gas was used.



SNOW ON AN ALASKAN BRIDGE

Medieval Piledriving Methods were resorted to in building the Sheepshead Bay automobile racetrack, Brooklyn, N. Y. The east side of the track lies in a salt marsh near the shores of Jamaica Bay, and it was necessary to put piles under the concrete floor-walls and column pedestals. The accompanying view shows one of the two piledrivers used. The job was



TWO-HORSEPOWER PILE DRIVER, SHEEPSHEAD BAY RACETRACK

one requiring as much speed as economy, and the two-horsepower piledriver shown filled the bill. The hammer was a round block of hard wood about 3 ft. long by 30 in. in diameter, raised by a block and fall and dropped by a man-operated tripping line. Only 6 or 8 ft. of penetration in muck was required. The track was described in "Engineering News," Aug. 19, 1915, p. 337.

New York City Water-Supply Distribution System—Excluding tunnels and masonry conduits there are 2800 mi. of mains of sizes varying from 4 to 60 in. in diameter, to which are connected 60,000 valves, 43,000 hydrants and over 360,000 services. Over 500,000,000 gal. daily are delivered through this system. In addition to the domestic water-supply system there are three separate high-pressure fire systems covering over 7200 acres, with 170 mi. of mains from 8 to 24 in. in diameter, on which are located 4100 hydrants and 6650 valves. To the existing distribution system there is to be added, probably in the present year, the great Catskill water-supply system, with its 17 mi. of rock tunnel, 22 feeding shafts, and 16 mi. of delivery mains, varying on land from 48 to 66 in. in diameter, and including two miles of 36-in. submerged pipe, connecting the boroughs of Brooklyn and Richmond.—W. W. Brush, Deputy Chief Engineer, Department of Water Supply, Gas & Electricity, New York City, in a paper presented to the American Water Works Association.

An Alaskan Snow Load—The accompanying view of a highway bridge heavily loaded with snow, taken in the winter of 1914, shows the two 60-ft. spruce-timber trusses of the Stewart Creek highway bridge on the Valdez-Fairbanks Road in Alaska. The view first appeared in "Professional Memoirs, Corps of Engineers, United States Army and Engineer Department at Large," for July-August, 1915, in an article by Capt. G. E. Edgerton entitled "Wooden and Combination Highway Bridges." In the article Captain Edgerton states that in rare instances in Alaska the snow load reaches as high as 60 lb. per sq.ft. on the floor of the bridges, but does not state whether or not this particular load is that maximum. It is evident from the view that not only the floor load but the snow load on the chords of such timber bridges must be taken into account in design. The article further states that "light snow weighs from 5 to 12 lb., and packed snow weighs from 15 to 45 lb. per cu.ft. Where a long series of weather observations is obtainable, the writer recommends that the maximum observed annual snowfall in inches be multiplied by 0.1 to obtain the proper snow load in pounds per square foot for the design of the bridges that are not to be regularly cleaned of snow. For bridges regularly cleaned, the maximum recorded snowfall in inches for a single storm should be multiplied by 0.9 to obtain a proper figure for the snow load in pounds per square foot."

Editorials

Lessons of Galveston's Storm

The Galveston seawall has been more fortunate than most engineering structures in the degree of appreciation it has received. In the popular mind it has shared somewhat the fame of the Holland dikes, for like those sturdy earth ridges it serves to protect human life and habitation against not only a battering but an engulfing sea. Merely as a matter of pride in their profession and its works engineers should be glad that the wall withstood its latest test, for the notoriety of its failure would have been as widespread as has been the fame of its success.

Aside from this just pride, though, engineers will recognize that the seawall at Galveston was not an exceedingly complicated piece of engineering construction and that given the type of beach and foundation available anything more than superficial damage to the wall was hardly to be expected. Apparently the only improvement that could be wished is an increase in height which would have prevented the overtopping and consequent scour of the boulevard and beach structures.

The wall admirably served its primary purpose—to keep the Gulf waves from battering down the structures that line the south shore of Galveston Island; but its secondary fnnction—to act as a dam to keep the high water out of the streets—was made successful only by the aid of the less spectacular but equally efficient grade elevation, which was in reality the most important work following the disaster of 1900. The fact that most of the flood damage seems to have been on the north side of the island where grades are still nearly as low as they were before 1900 points the way to future work.

Another feature of the latest Galveston disaster deserves pointing out. By far the greatest damage to structures was due to the wind. Against wind action engineers have only the most elementary of defenses. For the other destructive elements, fire and flood, preventive as well as protective measures can be taken, and these preventive measures are the most important part of the defense. Nothing has as yet been discovered that will prevent wind nor divert it from its destined path, and its vagaries of force and direction are all too little understood. In structures of sufficient importance the wind danger is properly considered, with more or less close approximation to pressures as they actually exist, but the vast majority of buildings in which the engineer has no part are designed with shameful disregard to the stresses to which they may be subjected in times of storm. It is only the rare occurrence of such winds as those which recently swept over Cincinnati and later over Galveston that saves such buildings from widespread destruction. Incidentally it may be remarked that the failure of the small house in wind or hurricane is practically always the result of insufficient anchorage. Almost any workmanlike framing will hold a house together if only it can be kept solid on its footings. And yet how many dwellings are anchored firmly to their cellar walls?

Pestilence and Floods

The notion that flood is followed swiftly by pestilence still persists in the minds of newspaper reporters and headliners, as has been shown since the recent Erie flood and Galveston storm. The notion appears to be founded more upon accumulation of dead bodies or perhaps a stoppage of sewers than upon possibly contaminated water and food supplies and the crowding together of homeless and helpless people.

To a large extent danger of a pestilence is popularly associated with a stench rising from decomposing organic matter rather than with the real danger of infection from the germs of communicable disease. This is not strange in view of long-continued persistence in the popular mind of the filth-theory of communicable disease.

It is a notable fact that epidemics of virulent communicable diseases, the real meaning of pestilence, did not follow the Johnstown, the original Galveston, the Austin (Penn.) nor the Dayton floods. However much this may have been the result of putting skilled sanitarians at once in control of the flood-swept districts, it is none the less true that the greatest dangers in each case were not because of the dead but because of the living.

This is the difference between the old and the new health-protective work: The first was directed against inert organic matter, which at worst could only cause a stench if left in place and not allowed to infect water and food used by human beings; the second goes swiftly and surely to the control of each infective human being, the destruction of emanations containing disease germs, and the protection of water and food from such germs as may possibly escape the utmost vigilance.

But for the moral effect upon the uninformed and the more or less indirect benefits arising from the destruction of relatively small amounts of infective matter, much of the strictly cleansing work considered so essential by the half-informed might be left undone after floods and other catastrophes causing loss of life with little increase in the number of cases of communicable disease. Nevertheless, sanitary authorities and the public alike are warranted in insisting upon rigid measures of cleanliness at such times, provided only they are carried out with a true conception of their health-protective efficiency and do not either directly or indirectly lessen the untiring vigilance with which health measures that count most should be conducted.

Sprinkling vinegar in a sick room was once considered a safeguard against virulent communicable disease. Scattering about some chemical reagent of unknown disinfecting value and many other measures practiced or advocated at the present day are equally futile against real dangers of pestilence. These must be combated with the weapons of modern sanitary science—the nature of which has not yet been sufficiently grasped by press reporters and headliners to fit the exigencies of news reports on great floods.

Fast Tunnel-Driving Records Under the Bonus System

The value of the bonus system of paying wages to secure rapid progress in driving tunnels is well shown by an article appearing elsewhere in this issue. This system was tried successfully at five recently built tunnels, where high-speed records were attained; and as it was the only important practice that was common to all these contracts, the records made can be attributed logically to the bonus system.

Deciding upon a reasonable base rate—that is, a rate of advance above which a bonus is to be paid—is the first essential step in applying this system to a job. Past performances and the law of continuity are the only accurate guides in arriving at a decision. A base rate may, in the light of altered conditions, become obviously unfair, in which case it should be changed. The bonus paid should not be excessive, but should be liberal enough to be a distinct incentive. A fair bonus system would be one which doubles wages if the rate of progress is doubled.

The principal saving effected through the bonus system of paying wages comes through shortening the time of driving a long tunnel and thus reducing the interest charges on the work. The sooner it becomes operative, the sooner it will begin to earn an income. There is also a saving to the contractor in reduction of charges for interest and depreciation on his plant and bond and insurance payments; while the saving in labor per linear foot of tunnel driven is also considerable. There is very little difference in the cost of supplies, as this tends to be nearly constant per linear foot of tunnel.

That men appreciate the opportunity to earn extra money is shown by the figures for the record month on the Mount Royal tunnel work at Montreal. The minimum or base rate was 14 ft. per day, making the total advance required in 31 days 434 ft. The rate of bonus was so proportioned that the men in the hole would just about double their wages if they doubled progress, no one outside of the tunnel participating in the bonus. The actual progress attained was 810 ft., which gave the workmen in the hole an 85% addition to their usual pay. The actual saving was only about 13% on the tunnel payroll, but was a saving of nearly 50% on the construction overhead, outside payrolls, interest, etc. Practically all the work on this contract was on a bonus basis.

An actual instance of the effect of the bonus system on the industry of the men is given by F. P. Brown, Chief Engineer of the Canadian Northern-Montreal Tunnel & Terminal Construction. Seven muckers were employed in the heading under normal conditions, three in advance throwing the muck back from under the bar and the four other men loading onto the cars direct. Mr. Brown says that normally a mucker thinks he is doing a good day's work when he shovels 7 cu.yd. of loose muck into a car, and that really this is all that can be expected of him. During the record month at the Mount Royal tunnel, however, the four muckers who were loading onto the cars handled up to 15 cu.yd. per man per shift, and averaged over 12 cu.yd. In order to appreciate that this is probably a mucking record, it must be remembered that nearly two hours were lost out of the eight in blasting, as two shots were fired regularly every shift. On this work a foreman never had to hire or discharge men during periods of rapid progress. If a man failed to do his share of the work, his comrades ran him out of the tunnel and brought in a better man to take his place.

In tunneling under the bonus system it is important to guard against a tendency toward carelessness on the part of the men in their endeavor to make high speeds. Where timbering is necessary, it should be regularly inspected.

Rainfall Troughs and Humps

In a brief article on long-time rainfall records in New York State written some time ago, but published in this issue of *Engineering News,* the author calls attention to the fact that we are "still in a 'trough' of a cycle of rainfall." The experience of the past six weeks and particularly of the past ten days might lead to the belief that, in the Eastern half of the country at least, this "trough" has become a hump. Of course students of meteorology know that abnormal weather conditions loom much larger at the time of their occurrence than they do in the records, but nevertheless the wide distribution, long continuance and occasional severity of the July and August rains of 1915 make them out of the ordinary.

Fortunately so far there have not been the coincidences of distribution and severity which bring about overtopped levees and terrible floods, but there have been any number of local "cloudbursts" and consequent floods varying in magnitude from the water-soaked cornfield to the flooded streets of Erie. Taken altogether the money and life loss has been as great as in one widespread flood.

From the data now available it would appear that one rainfall record has been broken—that is for amount of fall in 24 hr. At St. Petersburg, Fla., as noted in the news pages, a 24-hr. precipitation of 15.45 in. was recorded on Aug. 1-2. In Jewell, Md., on July 26, 1897, 14.75 in. of rain fell, and Galveston, Tex., has records of 12.16 in. in 12 hr. and 14 in. in 24 hr. on Oct. 22, 1913, and July 13, 1900, respectively. These seem to be the nearest approach in the United States to the recent Florida downpour, although the tropics could probably produce greater concentrated rains if the records were available. However, anyone who has seen an inch-an-hour rain can appreciate what a terrific display of the elements there is when a dozen inches are concentrated in as many hours, as was evidently the case at St. Petersburg. That city was extremely fortunate in not being located at the bottom of surrounding hills or else its destruction would have been certain.

Aiding Public when Roads Are Closed for Repair

A spirited correspondence has been going on in one of the New York morning papers over the difficulty motor tourists are having in finding their way through side roads, where a portion of the Albany Post Road has been closed for reconstruction. This happens to be a particularly popular road, and the objections of automobile owners here seem to be well founded, as they probably are in many similar cases. Every one who has toured over the roads of an unfamiliar country knows how annoying it is to be detoured from a good pavement to the ordinary earth road. Add to this a hilly topography with crooked roads and the aggravation is increased.

The highway engineer should never lose sight of the fact that he is a public officer, and his chief aim should be to give the public satisfaction so far as he can without interfering with the conscientious performance of his work as an engineer. Where annoyance to the traveler is inevitable, as must be the case where a portion of a through road is closed to traffic, let the annoyance be as slight as possible. Spend a few extra dollars and properly mark the detour throughout its whole length, or make it part of the contractor's duty to do so.

It might perhaps be well to go a little further where a detour is to be used for several months, and spend a few hundred dollars in making the worst portions of it a little more passable. This might be arranged with the local road officials, so that the improvements in the side roads could be made of permanent value.

The voluntary appraisal of a municipal water-works plant for the sake of readjusting rates on a financially sound basis and with due regard to justice to its consumers is sufficiently unusual to justify putting on record when it occurs. The water rates of the municipally owned plant of Alliance, Ohio, we are informed, were arrived at, as has so generally been the case in both municipal and private plants, on a hit-and-miss basis. The result, it is stated, is that the rates have not brought in sufficient revenue to pay the cost of operating the plant. The city officials have wisely decided to conduct a reform. In order to have the rates so adjusted that they will bring in sufficient revenue to pay operating expenses, and also make a proper return on the investment, the authorities have engaged Chester & Fleming, consulting engineers of Pittsburgh, Penn., to appraise the works.

It is not only municipally owned works, as intimated in the immediately preceding paragraph, that conduct some of their operations on a hit-or-miss plan and that finally realize this fact and call in outside engineers to put them right. The firm of engineers already named has just been engaged to operate the plant of the Upper Sandusky Water-Works Co., of Upper Sandusky, Ohio, and has for some time past been operating two other privately owned water-works. In each case these engineers are engaged on a yearly basis and are responsible for the entire management of a plant, including not only its operation, but also its financial affairs. Of course this is by no means the first time that engineers have been engaged to operate public utilities, but the practice might well be more common, especially as regards water-works, with profit to engineers and utilities alike.

Letters to the Editor

Patentees and Royalties

Sir—In the July 29, 1915, issue of *Engineering News* appears a letter from Daniel B. Luten in which he attempts to justify the bidding of a patentee on his own patented plans. The writer takes issue with some of Mr. Luten's statements in the matter.

Mr. Luten assumes that if "all bidders figure the same cost and profit, then the patentee would file the same bid as the others if he included the specified royalty for himself." This is perfectly obvious. However, on the qualification, "if he included the specified royalty for himself" hinges the whole argument. Suppose he doesn't?

A hypothetical case will be given and then followed by an actual case. Suppose a piece of work is to be done. Its estimated cost, plus profit and royalty, is $30,000. The royalty of 10% will be $3000. The cost and profit, as assumed by Mr. Luten, is the same in the case of all bidders. The bid will be: Cost, $23,478.26; 15% profit, $3521.74, royalty, $3000; total, $30,000. The patentee-contractor will receive $3000 regardless of who receives the award. But supposing the patentee-contractor is hungry for a job, and the work presents few difficulties so that the profit is fairly assured. Will the patentee-contractor let the other fellow have the work and take his $3000 royalty or will he "knock off" $500 from his figured royalty and take the job himself? In one case he will be assured of $3000, in the other case he has a good chance of making $6021.75, his estimated profit and $2500 royalty. Human nature being as it is, and contractors being the gamblers they are, it would be logical to assume that the patentee-contractor would take the chance at the $6021.75. While the patentee-contractor may not be so unfair as to take advantage of a competitor in this way, he might salve his ethical conscience by saying to himself: "Now, I've got that $3000 'edge' on the other fellows, I'll cut $500 off my profit." The writer is not a pessimist in the matter of the honesty of his fellow-men, but gives the foregoing merely as an example of what might happen. He will now give an example of what did happen.

A year or so ago the writer submitted a bid on a small arch span in competition with the representative of the owner of the numerous cost-saving patents on this type of structure. The two submitted bids were $2265 and $2275, the writer's being the lower by $10. Both bids were rejected, and the patentee-contractor, as he will be called, was awarded the job the same day at a private letting for $1975, a reduction of $300 from his original bid.

The writer's bid was as follows: Cost, $1698.75; 20% profit, $339.75; royalty, $226.50; total, $2265.

The patentee-contractor's bid, figured on the same basis would have been: Cost, $1706.25; 20% profit, $341.25; royalty, $227.50; total, $2275.

The limit at which work may be let at private letting is $2000. The bid was let at $1975. The queries arise: Did the patentee-contractor, whose profit and royalty amounted to $568.75, (?) deduct the $300 from his profit, leaving himself but $41.25, or did he eliminate the royalty entirely and deduct $72.50 from his profit? What would have happened had the low bidder deducted $300 in order to get the work? The answer to the first query is that it makes no difference which he did. The answer to the second is that he would not have done it. In this

case the patentee-contractor most assuredly had an advantage—and used it!

The writer agrees with Mr. Luten, however, that such lettings are fair, if certain qualifications are present. These qualifications are that all parties to the letting be fair and honorable and that the patent have merit. Mr. Luten mentions the attitude of engineers and contractors toward patented plans and intimates that the attitude is unfavorable. He fails to make any distinction between the relative values of patents. Toward patents having intrinsic merit and value engineers and others are fair and have no disinclination toward paying royalties; indeed are very anxious to do so if they are enabled to save money thereby. Unfortunately for the profession, however, a great many patents are issued which have no value whatever from an engineering standpoint.

R. C. HARDMAN.

Colorado City, Colo., Aug. 14, 1915.

Mortar Cushions for Granite-Block Pavements

Sir—While agreeing with what you say about the general unwisdom of rashly departing from well-established engineering practice, in your editorial of Aug. 5, 1915, on "Mortar Beds for Brick and Stone Pavements," I am of the opinion that this is one of the cases where such departure is warranted.

The arguments you present in favor of sand-cushions are old and standard, but are open to challenge. You admit that the sand cushion is not elastic, but claim that it has some "resiliency" under travel. Now resiliency and elasticity are practically synonomous terms; but it is made plain that by resiliency you mean that kind of yield which, in sand, is better expressed by mobility—that is, shifting, a property or condition that engineers agree is deleterious if not fatal to a pavement.

The fact doubtless is that a body of sand, thoroughly compressed in a confined space, is absolutely rigid and can have only such elasticity as is possessed by the individual grains. This elasticity probably differs little from that of a granite block itself and is too infinitesimal to have any effect on travel over the street. The sand cushion under a properly constructed granite-block pavement doubtless becomes in time as rigid as the concrete foundation, otherwise constant yielding must in time either cause the sand to disappear or disturb the pavement.

I have given a good deal of attention to this matter and was once, many years ago, so rash as to recommend for a highway a brick pavement with the brick set flatwise on a bed of soft, rich mortar, the joints to be filled with the same mortar (and, by the way, was promptly "turned down" by both the local engineer and the brick manufacturers). Hence, I take a lively interest in the present discussion and experiments, which I believe will in time lead to the abandonment of sand cushions.

We now build our pavements with a concrete foundation to carry the loads and a wearing surface to resist wear, interposing between the two a bed of sand possessing no strength for either function. Why not rather unite foundation and wearing surface in a way to utilize their combined strength by making the structure monolithic? The obvious way to do this is by setting the brick or stone in a rich mortar bed placed on the green concrete foundation and grouting the joints, all the work being done before the cement in any part begins to set.

This would enable us materially to decrease the thickness of both the foundation and the wearing surface. The full thickness of the whole pavement would be utilized to carry loads. Neither granite blocks nor bricks require the depth now common in practice to take care of the surface wear upon them.

A granite block or a brick 3 in. in depth will give about as much service as one 7 in. in depth if it can be securely and rigidly held in place on the street; and if supported on a 4-in. concrete foundation, the pavement would have about the same load capacity as one built with a 6-in. concrete foundation, the usual sand cushion, and 6-in. granite blocks—barring the possible arch action of the blocks.

Such a construction would obviously reduce the first cost, and there is no reason to believe it would increase the cost of maintenance. The return in service rendered per dollar invested would certainly be materially increased. There is not, so far as I am aware, any reliable evidence that such a pavement would be appreciably less "resilient" or less easy upon traffic than those with sand cushions.

Years ago, the brick manufacturers asserted that brick laid on their sides would not wear well because of the direction of the "grain," due to the method of manufacturing them. It is not clear how a vitrified brick can have much grain, but in any event it is now claimed that this alleged difficulty has been overcome.

S. WHINERY.

New York City, Aug. 9, 1915.

[The questionable "resiliency" of a sand cushion could be very easily demonstrated or not by breaking cubes of stone under impact, one test to be with the cubes resting on a sand bed and the other test with the block set in mortar or concrete. No one can deny that however closely coarse sand is packed, there is still a large percentage of voids in the material, while certainly a "rich mortar" ought not to have many voids. A material with 30 per cent. voids has probably more cushioning effect than a rigid solid.—Editor.]

NOTES AND QUERIES

Louisville Bascule Bridge—The novel lever lift bascule bridge over the Louisville & Portland Canal at Louisville, Ky., described in "Engineering News," July 15, 1915, p. 123, was fabricated by the Penn Bridge Co., of Beaver Falls, Penn. It was erected by the Middle States Construction Co., of Columbus, Ohio.

Water-Works Bill Receipting and Clipping Machine—J. Walter Ackerman, Chief Engineer and Superintendent of Water-Works, Auburn, N. Y., informs "Engineering News" that he at one time used a machine for receipting the bill and coupon or stub of a postal-card form of water-works bill. This machine perforated the date on both the coupon and the stub and then cut the two apart. Mr. Ackerman desires to get a machine which will do this and which in addition will deposit the coupon in a receptacle provided for the purpose. His original machine was made by the B. F. Cummings Co. Mr. Ackerman has recently been using a rubber stamp for dating and receipting his water-rate bills. He finds that sametimes the date made by the rubber stamp becomes obliterated. He adds, as of possible interest to readers of "Engineering News," that "after his postal card form of water bill has been receipted and the coupon clipped off, a stamp-affixing machine is used to place a one-cent postage stamp over the original stamp on the card, after which the postal is mailed back to the consumer as a receipt."

Shipping Piers in Southwark District of Philadelphia

SYNOPSIS—Two shipping piers 550 ft. long, 170 ft. wide with 200-ft. docks. In center of Philadelphia's shipping district. Concrete substructures on timber piles. Steel and concrete superstructure.

The city of Philadelphia is now completing two large shipping piers on the Delaware River within one mile of Market St., the center of the city's active waterfront, and are known as the Southwark piers, and are designated as Nos. 38 and 40 South Wharves.

The city is about to embark on a very large harbor development, in which is contemplated the construction of a series of twelve or more large piers with uptodate mechanical equipment and direct railroad connections. These piers will range in size from 300 ft. in width to 1200 ft. in length, and will be located at the south end of the city's waterfront, in a district not now adequately developed on account of adverse railroad connections. The

FIG. 1. RECENT VIEW OF HEADHOUSES OF SOUTHWARK PIERS FROM MARGINAL STREET

within easy teaming distance of the principal warehouse district. These piers are distinguished by their exceptional design and freight-handling equipment. On account of the old district in which they are located they new plan, however, contemplates also the entire revision of the railway yard and tracks at the south end of the city, so that ample railway facilities will be afforded to the pier district, and, in addition, a better development

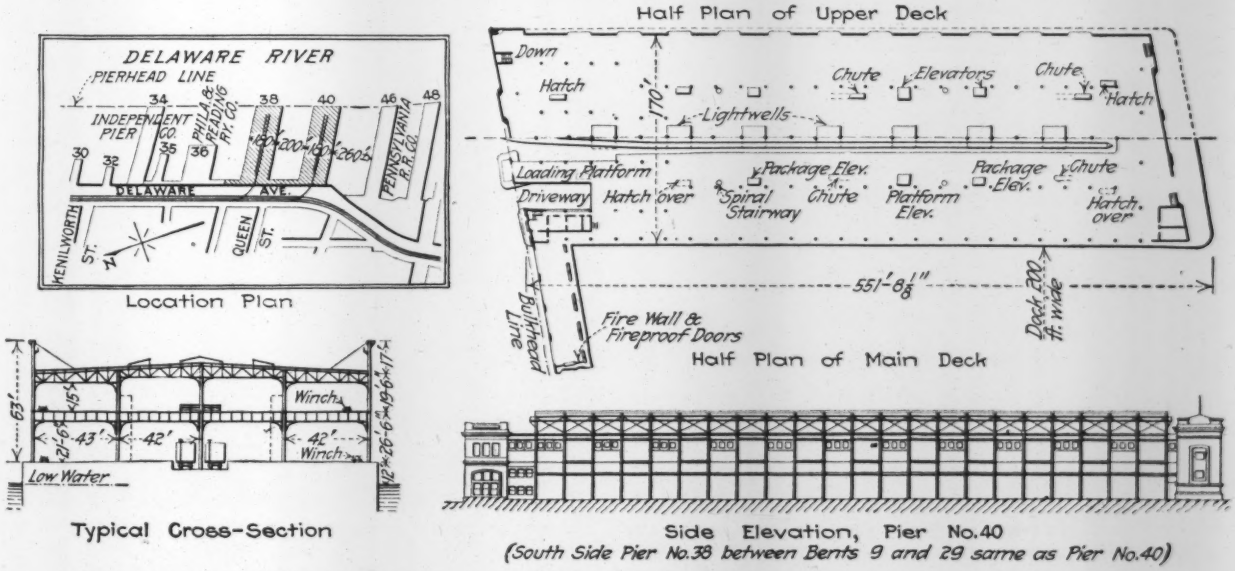


FIG. 2. GENERAL DETAILS OF SOUTHWARK PIERS NOW NEARING COMPLETION ON DELAWARE RIVER

of the railway terminals will be had. These piers, one of which has just been started, are entirely separate from the Southwark piers, which are in the part of the waterfront already under development and which are intended not only for permanent future use but to provide ample steamship accommodations until the new pier area is completed.

Piers 38 and 40 are each 170 ft. wide by 550 ft. long, with docks 200 ft. in width flanking each pier (Fig. 2) on a timber platform with concrete retaining walls on the outside; (3) a solid earth-fill pier with outside retaining wall and the interior columns carried on timber piles; (4) a solid earth fill of the same type, except that concrete piles were substituted; (5) a timber and concrete design in which the pedestals for the pier sheds were of concrete and all the rest of the substructure timber; (6) a type similar to the first one named, except that instead of a concrete cross-wall, extending clear from the

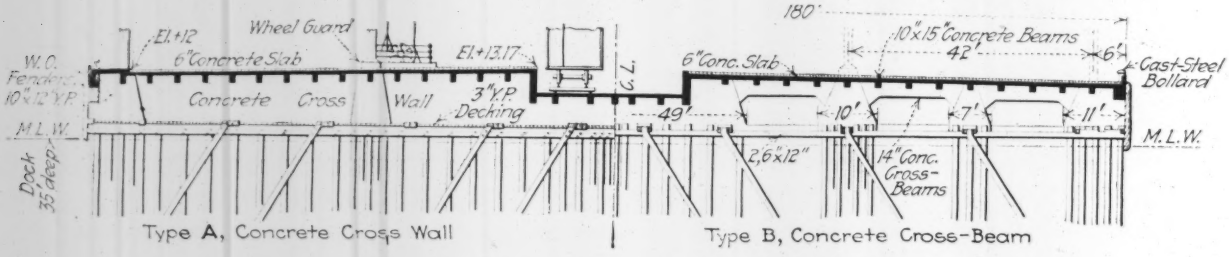


FIG. 3. TWO ALTERNATE DESIGNS FOR SOUTHWARK PIERS
Type A was used for Pier 38 and Type B for Pier 40

The superstructures of the two piers are identical, comprising two-story steel-and-concrete sheds, with inshore and outshore headhouses for office purposes and a line of bulkhead shed 30 ft. wide paralleling the marginal street and connecting the two piers. The headhouses are covered with concrete of pleasing architectural design.

The substructures to the new piers are practically identical, differing only in a minor particular. In 1914 the Department of Docks, Wharves and Ferries made a number of alternate designs for the substructure of the then proposed Southwark piers and called for bids on the different types. These designs and the bids were described in *Engineering News* of May 28, 1914, p. 1212. All the substructures had timber piles for footings, the variation being in the method of carrying the pier above the tops of the timber piles, which were cut off at mean low water. The different designs comprised (1) a concrete cross-wall carrying a concrete floor; (2) an earth fill floor of the pier to the cappings of the piles, concrete pedestals under the columns rested on pile cappings and were connected by reinforced-concrete beams.

After going over the submitted bids it was decided to build Pier 38 after the first-named type—that is, with concrete cross-walls—and Pier 40 after the last named—that is, of concrete cross-beams. A section through these two alternate designs is shown in Fig. 3. Fig. 4 shows the details of the design of the substructure of Pier 40—the one with concrete cross-beams. Except for the detailed differences shown in the comparative designs in Fig. 3, the details of Pier 38 are the same as those shown for Pier 40.

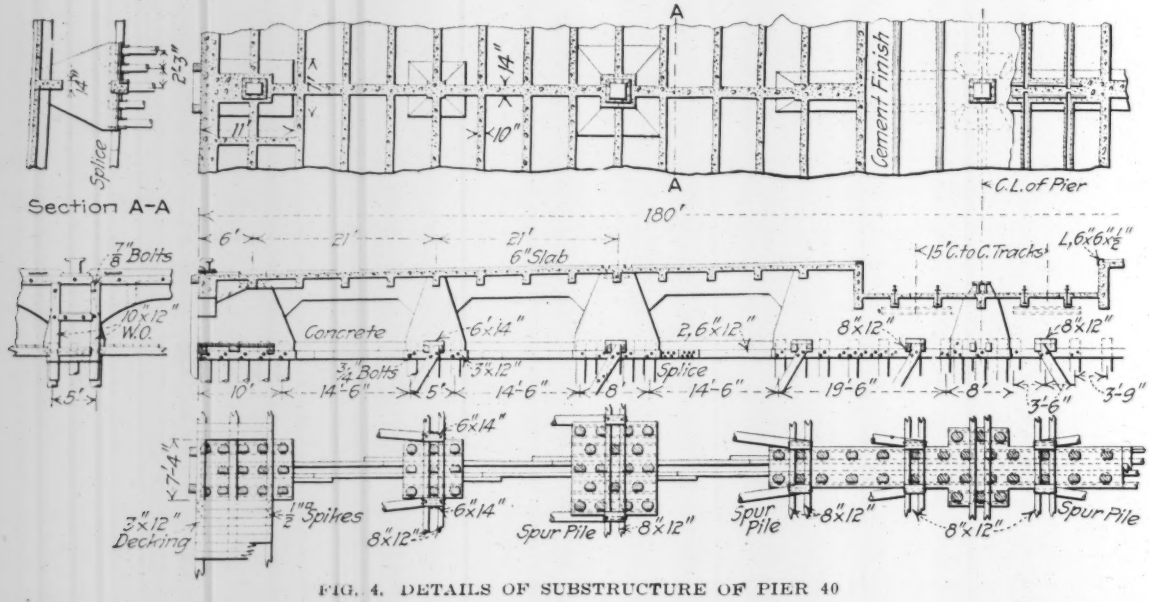


FIG. 4. DETAILS OF SUBSTRUCTURE OF PIER 40

The substructures consist of groups of straight and battered timber piles carrying longitudinal and transverse timbers tying the pile groups together. On top of this is poured the concrete structure with a reinforced-concrete slab floor, on which is erected the steelwork of the shed.

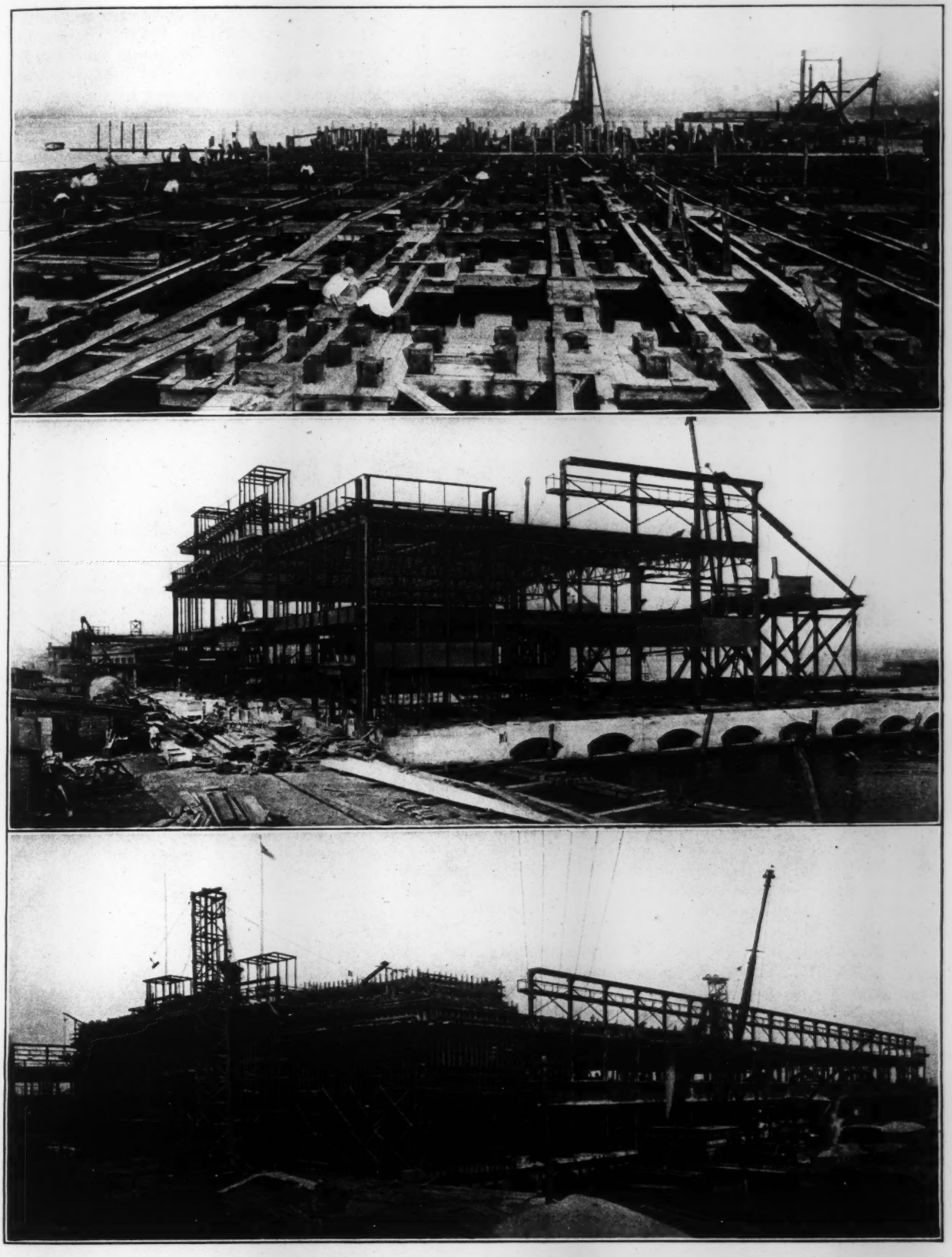


FIG. 5. VIEWS OF SOUTHWARK PIERS, DELAWARE RIVER, PHILADELPHIA, DURING CONSTRUCTION

5-A—Timber piles and connecting beams with cross platforms on which cross walls were laid. 5-B—Street headhouse steel under erection. 5-C—Steelwork erected; concrete covering to street headhouse being placed

In the center of the piers are two depressed railroad tracks, which are connected with the tracks of the Philadelphia Belt Line R.R., from which any of the trunk lines entering the city can be reached.

The first floor of the shed is designed for loadings of 600 lb. per sq.ft., and the second floor for a loading of 400 lb. per sq.ft. Special attention has been given to loading and unloading facilities, and the inshore end of the car pit has been enlarged for this purpose. This space, together with that along the bulkhead shed, will permit from 40 to 50 teams to be unloaded simultaneously on each pier on the platforms at the level of the dray body without interference with cargo-handling operations on the pier decks. The main deck is raised about 3 ft. above the street level, in order to bring it to the level of the floors of the cars and the central depressed pit, and is approached by short ramps inside the pier entrances leading from the marginal street.

The mechanical cargo-handling equipment of the piers will include electrically operated elevators of both the platform and the continuously operating cage type, cargo chutes and hatches for transferring freight between the upper and lower decks, and portable electric winches with portable controllers. Fig. 5A is a view of the substructure timbering in Pier 38 before the placing of the concrete. Fig. 5B shows the Pier 38 substructure completed, with the inshore headhouse steelwork under way, and Fig. 5C shows the same pier with the falsework for the inshore headhouse concrete in place. This was rather peculiar, in that the entire falsework was placed for the two stories at one time, the concrete protection not being carried up by stories, as is generally the case in reinforced-concrete buildings. The view in Fig. 1 shows the present condition of the improvements as seen from the marginal street. Pier 40 is in the foreground.

The new Southwark piers are being built by the city of Philadelphia under the direction of the Department of Wharves, Docks and Ferries, of which John Meigs is director. The contract for the entire work is held by the Snare & Triest Co., of New York City.

Galveston Survives Hurricane

SYNOPSIS—Gulf hurricane sweeps over Gulf of Mexico, causing damage all along shore, mainly at Galveston. Protective measures there prove successful. Although temporarily undermined at two points, the gaps beneath the seawall were filled in by the receding tide and the wall left intact. Backfill and pavements damaged by water that rushed through the gap and overtopped the wall. The 8200 ft. of earth approach to causeway demolished, but the 2500 ft. of reinforced-concrete arches and the 100-ft. rolling-lift steel bridge practically undamaged.

The city of Galveston, Tex., was subjected on Aug. 16 and 17 to one of the most violent wind and rain storms in the history of that hurricane-swept city. Thanks to the precautions in the way of seawall and grade elevation taken subsequent to the disastrous storm of 1900, the loss of life and damage to property was comparatively small in the city although the loss of life along the Gulf shore will probably total 250.

The storm broke over the city at 5 o'clock Monday afternoon, Aug. 16. The wind, it was estimated, had attained a velocity of about 60 mi. per hr. This was the wind from the north, and it continued to blow the water in from the bay, flooding the streets and cellars. By 9 o'clock the water was from 4 to 6 ft. deep in Market St.

The wind, still blowing from the north, continued to increase in velocity until about 2 a.m., Aug. 17. At that hour, it is estimated, it had reached a velocity of 90 mi. per hr. Meanwhile it had veered from the north around to the east and then came directly out of the south. This south wind, sending in the waves from the gulf to break against the seawall, damaged the southern part of the city and the boulevard.

The center of the storm passed west of Houston, Tex.,[1] at 4:35 a.m., Tuesday, when the barometer read 28.21 in., which probably is the lowest on record in the United States. The wind was violent, in pounding gusts, and was accompanied with driving rain on Monday and Tuesday. The maximum velocity recorded at Houston was 80 mi. per hr. but was probably higher at times as the anemometer failed to record for an hour and a half when the wind was most violent. The rainfall at Houston totaled 6.50 in. for the storm and 9.09 in. for the entire period of rainfall. The storm passed northwest, being central near Bryan, Tex., at 9 a.m., Wednesday, and curved northeast, giving torrential rains in eastern Texas and Missouri, where flood trouble also was experienced later in the week. The storm reached New York State on Sunday, Aug. 22, causing small floods and resulting in minor damage there.

For two or three days Galveston, where the greatest damage was done, was cut off from communication. *Engineering News* immediately wired instructions to a representative to try to reach Galveston. This correspondent did reach there in time to prepare the article which follows this general description of the storm itself.

The damage was not confined to Galveston, but was distributed well along the Gulf coast. On account, however, of the history of Galveston and its island, and the efforts made to protect it against such storms, the conditions there are of most interest to engineers.

History of Storms

The city of Galveston is located on the eastern end of Galveston Island. The original elevation of the ground was in many places but little above ordinary high tide of the Gulf, and at extraordinarily high tides the water came into the streets in many places. This condition was helped somewhat by filling up in low places.

About 1875 a general system of levels was taken through the city, and grades to the streets were established. The elevation of the natural surface of the ground in the streets was nowhere greater than 8.9 ft., and the average elevation from 6th to 39th St. is about 5.8 ft. The average elevation of the ground west of 39th St. is much less.

[1] Information by wire from R. Bunnemeyer, United States Weather Bureau observer, Houston, Tex.

The higher parts of the city are generally above overflows during storms, and it is only during phenomenal storms that the city is completely or to a great extent submerged. The district, however, is subject to these phenomenal storms. Within the memory of those living they have occurred in 1834, 1837, 1847, 1854, 1867, 1875, 1866, 1900, and 1909[2]; that is, at intervals of never less than

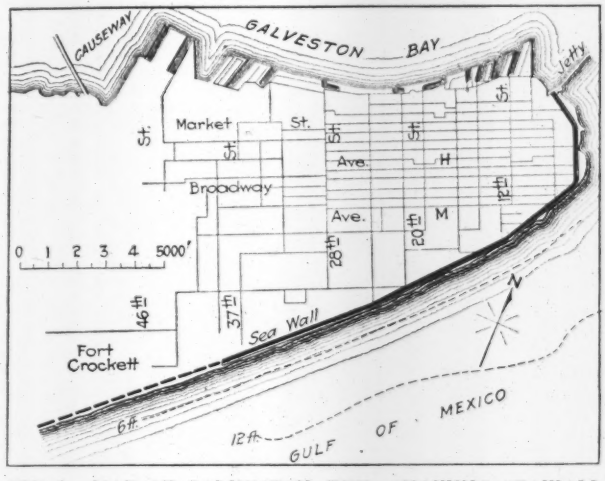


FIG. 1. MAP OF GALVESTON, TEX., SHOWING SEAWALL

Main seawall shown by black line along south and east coast. Dotted line at west end of city is the wall built later by the Federal Government.

three years and never more than 13 years. In early storms the actual height of the water is of course not known, but in the devastating storm of 1900, which caused the practical destruction of the city and the deaths of about 5000 people, the water reached a height far exceeding any previous record. At that time, at 8th St. and Ave. B the elevation of the water was 15.7 ft., and at the Southern Pacific wharf near the Galveston end of the causeway (see map), the lowest water on the island, the gage read about 10 ft. above mean Gulf level. At that time the wind reached a velocity of at least 100 mi. per hr., and the damage was due equally to the wind and the force of the water, which drove into the city in huge waves.

Protective Measures

Following the 1900 storm the abandonment of the city site was at first considered, but inasmuch as the Government had settled upon Galveston as the one harbor in the Gulf west of New Orleans which warranted improvements (on account of the possible great working depth there), it was decided to rebuild and to protect the island against similar storms. This protection, which was later undertaken, was designed by Alfred Noble, H. C. Ripley, and Gen. Henry M. Robert. It consisted in its final form of a seawall around the south end of the city from the jetty extending out into Galveston Bay at the north, to the beginning of the Government reservation at 39th St. on the Gulf shore. This wall was 3⅓ mi. long and was afterward continued about 1½ mi. by the Federal Government along its reservation at Fort Crockett.

A general section through the wall is shown in Fig. 2. It consists, as will be noted there, of a gravity section concrete wall, reinforced by a single line of rods near the curved face, resting on piles and protected in front by riprap and a line of sheetpiling. This wall was originally intended to be protected at its rear by an earth fill of a width of 100 ft.; that is, for the width of the Boulevard which paralleled it. A later project, however, involved the filling behind the wall for such a distance as would raise the grade from 10 to 16 ft. over the entire island. Only a part of this project was completed, but this part was sufficient to elevate the south half of the island on a slope which increased from the previous elevation at the Galveston Bay side up to the top of the gravity wall. The filling amounted to about 12,000,000 cu.yd. and cost approximately $2,000,000. It was made by hydraulic dredges pumping from a canal dug along the inside of the city limits of Galveston. It was finished about 1906.

Galveston Causeway

The other important engineering structure in Galveston is the famous Galveston Causeway. This carries railways and a roadway across the two miles of Galveston Bay from the island to the mainland. It was started in April, 1909, and opened in May, 1912. The causeway consists of 3696 ft. of earth fill approach on the mainland, 2455 ft. of reinforced-concrete viaduct made up of twenty-eight 70-ft. arches, and 4523 ft. of earth fill on the Galveston side. The center of the viaduct contains a 100-ft. Scherzer rolling lift bridge. The concrete viaduct is 66 ft. wide and the causeway approaches are 119 ft. wide, carrying two railroad tracks, two elevated railroad tracks and a 40-ft. roadway. The structure was designed by the Concrete Steel Engineering Co., of New York City, and built by the A. M. Blodgett Construction Co. The concrete-arch section was of the regular reinforced-concrete arch design, with the piers founded on timber piles. The approaches were sand-and-gravel fills and for most of the length had the slopes protected by concrete slabs, resting on concrete sheetpiling.

As noted in Mr. Babbitt's article, which follows, the main damage in Galveston consisted of the destruction of buildings by wind and, along the seawall, by the force of the overtopping waves, and the washing out of the causeway approaches. The total number of killed has not yet

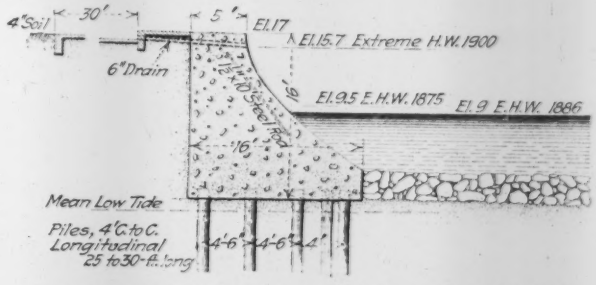
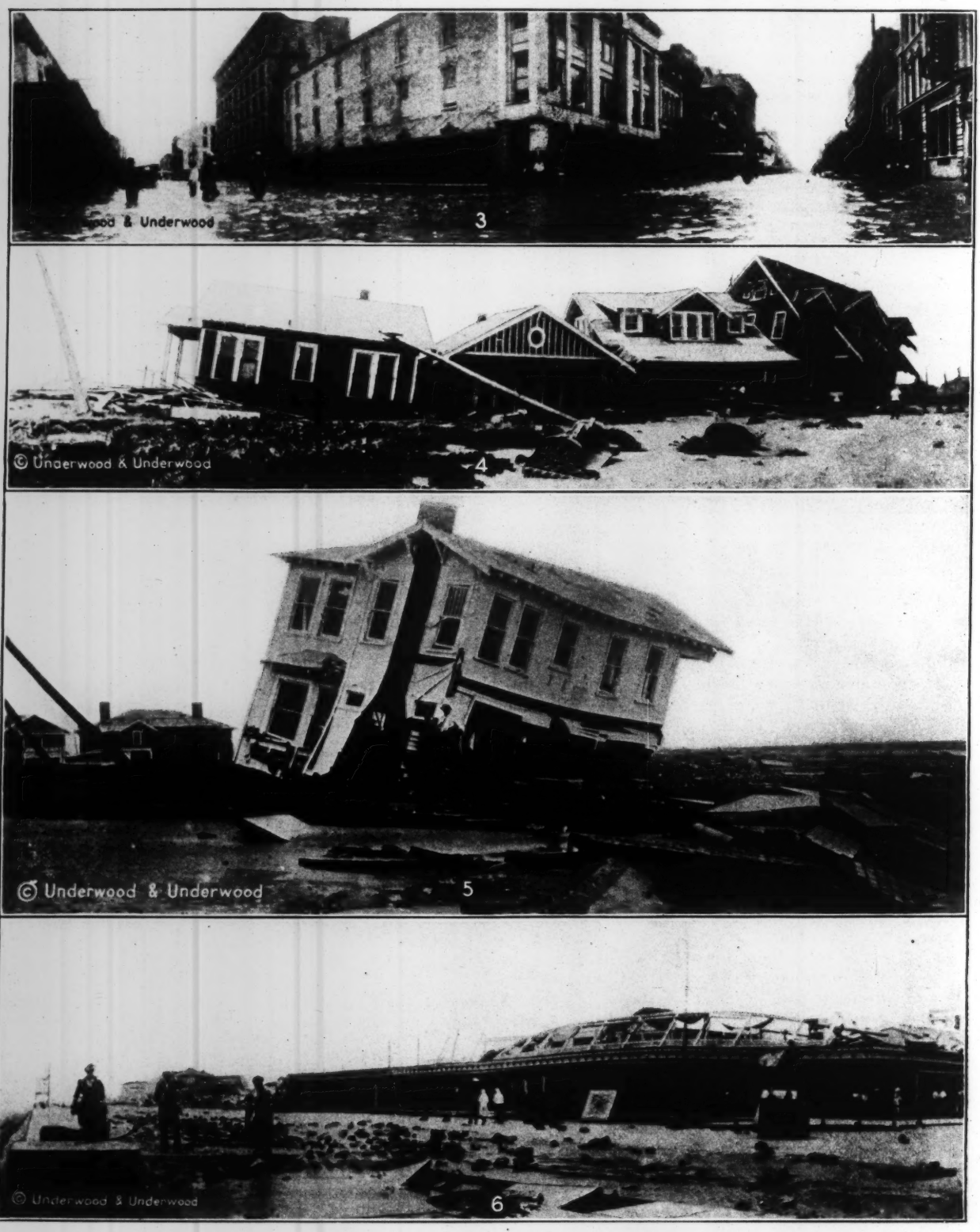


FIG. 2. SECTION THROUGH GALVESTON SEAWALL

been ascertained, but it promises to be very small. Practically the only injuries or deaths were in the frame buildings which were blown down. The water of the Gulf in backing up flooded the northern part of the city where filling had not been made, but this flood had not the force of the 1900 wave, and was not particularly destructive. The main carrying the city water-supply across the causeway was wrecked with that structure and was not replaced until Aug. 24, when an 8-in. temporary main was laid.

[2]The 1909 flood, while second in intensity to those of 1900 and 1915, caused very little damage.



FIGS. 3 TO 6. VIEWS IN GALVESTON AFTER THE WINDSTORM OF AUG. 16

Fig. 3—23d and Market St., on north side of island, showing flood backed in from Galveston Bay. Fig. 4—Houses torn from foundation at 33d St. and Boulevard on the seawall. Fig. 5—House at 14th St. and seawall. Débris in right foreground from paved boulevard paralleling seawall. Fig. 6—Looking along seawall at 23d St. Buildings in background blown down by wind. Note stone monuments against curb near house torn up from top of seawall by force of waves

Effect of Galveston Storm on Seawall and Causeway

By R. P. Babbitt*

By wire from Houston, Tex., after special investigation at Galveston

Galveston's seawall is intact but its causeway to the mainland is destroyed, except for the reinforced-concrete arched portion and the rolling-lift steel bridge.

The West India hurricane which swooped down upon the Gulf Coast on Aug. 16 was the third greatest storm in its history. The wind came from the northeast for 30 hours with but little abatement. Had it not been for the seawall there would now be only a heap of ruins to mark the site of this great Texas seaport.

This magnificent wall, extending for approximately three miles along the Gulf front of the city, was impregnable to the fury of the waves which beat against it but were baffled at every point. In spite of this tremendous assault the seawall stands today as perfect a structure as the day it was completed. Not a crack or defect resulted from the furious attack of the forces of nature.

Seawall Temporarily Undermined

At two places only did the waters of the Gulf pierce beneath the wall—temporarily and without injury to the wall itself. One of these was near Fort Crockett, at the southern end of the wall; the other, at the northern end, between Broadway and 21st St. At each of these points the Gulf waters undermined the structure, hollowing out the sand until the piles upon which the wall rested were exposed to view. But with the return of the tide these gaps were refilled with sand and not a fault developed in the wall. Although the seawall was not damaged, some injury was caused by the material which was swept through the two temporary gaps beneath the wall.

Effect of Water Overtopping Seawall

Further injury was wrought to the backfill and pavements behind the seawall by the tremendous masses of water that were projected over the seawall to a distance of 200 ft. or more. Where this occurred pavements were undermined and the backfill was washed out, forming a wide stretch of beach behind the wall.

It should be explained, in passing, that upon the completion of the seawall the United States Government backfilled that portion of the structure for the building of which it was responsible. Unfortunately, as recent events have proved, the Galveston authorities failed to continue the backfill as far cityward as was needed, so that when the waters of the Gulf were driven before the hurricane of Aug. 16 they overleaped the wall, tore up the backfill, and worked havoc with the streets and boulevards in the vicinity.

Where there was sufficient backfill to prevent the overfalling water from getting under it, as at 25th St., damage to the pavement was prevented and the boulevard is in surprisingly good condition.

The damage to backfill and pavement caused by the water which passed through one of the gaps beneath the wall was studied by the writer at 23rd St. Here the backfill was washed out and silt swept in by the inrushing water, thus forming a beach several hundred feet wide. So far as could be discovered by an examination the seawall here, on both the Gulf and the island side, is in as perfect condition today as when completed. At 19th St. the seawall was also found intact, but the overfall of the water undermined the pavement and caused it to settle.

Complete the Seawall and Raise the City

One great lesson was demonstrated to the Galveston people, and that is the necessity of completing the seawall about the entire city front, then raising the level of the city above the level of the top of the seawall. Had Galveston been above the level of the present seawall during the hurricane of Aug. 16, the water damage to the city would have been very slight, for the overfall would then have drained back into the bay on the one hand and into the Gulf on the other, and the water damage caused in the central portion of the city would have been comparatively slight. The water damage to the business district of the city was surprisingly small considering the violence and duration of the storm.

In 1900 the waves from the Gulf swept the city, which then had no seawall to protect it. This caused the tremendous loss of life and property which occurred at that time. During the recent storm the water that rose in the city from the bay was caused by the pressure of the waters from the Gulf. There was no wave motion in the streets of the city in 1915, as was the case in 1900.

It is probable that the seawall will be extended, entirely encircling the city, and the city raised above the level of the top of the wall.

Earth-Fill Causeway Destroyed

Although the seawall, or outer line of defense, saved the city from destruction and proved its own integrity, the city's only line of communication with the mainland was not only broken but in larger part destroyed. The only portion remaining intact is the 2450 ft. of 28 reinforced-concrete arches, 70-ft. span, and the rolling-lift steel bridge of 100-ft. span in the center of the arches. This part of the causeway was practically undamaged.

Had the entire causeway been of reinforced concrete Galveston would not have been cut off from the mainland and her water and food supply would not have been reduced to the low point that was brought about by the demolition of the two end sections.

From this permanent structure a "protected roadway" extended eastward approximately 37 ft. to the mainland and westward for about 4500 ft. to Galveston Island. Both these sections were demolished.

These protected roadways consisted of sand-and-gravel fills 119 ft. wide on top, with their side slopes protected by 6-in. concrete slabs. The toes of this slope paving rested, on each side, upon tongue-and-grooved reinforced-concrete piling. After the piles were driven the grooves were poured full of concrete, thus forming a solid wall. The concrete sheetpiling was then capped.

So far as could be discovered, the lines of concrete piling were intact at every point, but the sand-and-gravel fill was scooped out by the action of the water, letting down the concrete slabs.

As has been said, the reinforced-concrete arch section of the causeway is practically uninjured. Not a break was found in piers or in arches.

*Civil Engineer, Dallas, Tex.

News of the Engineering World

Progress on 46th St. Piers

The view on this page shows a recent stage in the construction of the 46th St. piers on the North River, New York City. The inside of the huge coffer-dam is shown with the drilling and blasting going on in the rock bottom of the river. This coffer-dam, as described in *Engineering News* of July 23, 1914, p. 212, consists of a series of connected cylinders made up of interlocking steel sheet-piles, the cylinders being filled with earth so as to form a solid wall. In addition the inside and the outside of this wall are heavily backed with riprap, as shown in the view.

INSIDE OF THE 60-FT. COFFER-DAM HOLDING THE NORTH RIVER AT 46TH ST., NEW YORK CITY

The dam is now holding back about 60 ft. of water, with comparatively little leakage.

Flood at St. Louis

The damage done by the recent storm in and around the Gulf was largely localized and resulted almost entirely from the wind. In Missouri, however, the concentrated rain of Aug. 19-20 caused considerable damage in and around St. Louis. The record for the 24 hr. between 7 p.m., Aug. 19, and 7 p.m., Aug. 20, shows a maximum in St. Louis of 9.35 in. with a wind velocity of 48 mi. per hr. This is a rainfall nearly double any previous 24 hr. record in St. Louis. The fall was fairly steady and the intensity was not exceptional. Consequently, little more sewer trouble was reported than from the heavy short rains during the summer.

The River des Peres, which runs through the western part of the city of St. Louis, however, caused considerable damage. This river, which has a drainage area of about 110 sq.mi., practically a quarter of which is in St. Louis itself, broke all records for high water, overtopping nearly all of its crossing bridges except the high concrete arch at Clayton Road. All street-car traffic and trains to the west were suspended, and the driveway on Clayton Road was for over 12 hr. the only communication line across the stream. In the West End residence district of St. Louis the river flooded from half a mile to a mile wide, tore up some wood-block pavements and washed out several of the small street bridges. Although plans for controlling the stream had been prepared, no money had ever been appropriated for the work. It is thought that now the stream control will be undertaken at a cost of about $5,000,000.

Later in the week, the Meramec River, which empties into the Mississippi just south of St. Louis, was in flood because of the same rainstorm. This river rose many feet and spread over a considerable area of the lowland, causing some damage to summer residences on its banks.

Day's Rainfall of Over 15 In.

A precipitation of 15.45 in. for the 24 hr. ending 5 p.m., Aug. 2, was recorded in St. Petersburg, Fla., on the rain-gage maintained by the *Evening Independent* for the use of the United States Weather Bureau. The first phase

of the storm lasted only a few minutes. The heavy precipitation commenced at 8 p.m., Aug. 1, and ended about 1 a.m. the following day. From that time to 1:30 p.m. there were only intermittent light showers. L. W. Brown, who furnishes us with this information, states that it is safe to say that not much more than ½ in. of rain fell outside the 12-hr. period from 8 p.m. to 8 a.m. There was practically no wind accompanying the rain. Periodic readings were not taken.

The greatest damage in the city was done to brick paving, especially where the paving ended along the waterfront; here it was undermined to a considerable extent. Similar damage was done to the brick paving of a boulevard surrounding Mirror Lake—a 10-acre body of water located in the north-central part of the town. There was a rise in the lake during the night of about 7 ft. Drain pipes from the lake and storm sewers in the streets were far too small to handle the runoff. Booker Creek, normally a small stream flowing through the southwestern part of the town, went out of banks, and washed out approaches to several concrete bridges, and carried away some of the weaker bridges crossing it. Portions of concrete wall along this stream in the Roser Park section were washed out. The total damage is estimated at $25,000.

Outside of St. Petersburg the roads were badly washed and most of the small bridges were carried out. These roads were constructed principally of marl—which is really little more than a good stiff clay. They are not being permanently repaired, as it is the intention to pave them with brick during the next few months.

The telephone system was crippled for one day, mainly through trouble with the underground cables. Interurban car lines were also out of operation for one day, on account of washed-out bridges. The traffic on two railways, touching at this town, was stopped for about the same length of time.

The rain was general in Florida, but particularly severe in the southern portion of the state.

Projected Automobile Speedway for Louisville, Ky.

Plans are being prepared by a company organized by George L. Martin, 237 South Fifth St., Louisville, Ky., for the construction of a 2-mi. automobile racetrack near the present Douglas Park horse-racing track. The new speedway is to have a plank floor on concrete walls, according to present plans. Four grandstands with a seating capacity of 15,000 each are to be built.

About 300 acres of land has been purchased, most of which is low-lying and will require considerable drainage, but little excavation. A topographic survey of the plot has been made by Stonestreet & Ford, 414 Louisville Trust Building, Louisville, engineers for the speedway company. The cost of all improvements is estimated at between $250,000 and $300,000. It is intended to have the track completed by the spring of 1916.

Harlem River Draw Replaced

The accompanying view shows a stage in the process of floating out the old drawbridge of the Third Ave. and Second Ave. Elevated R.R. bridge across the Harlem River in New York City and floating in the new double-deck drawbridge which will carry four tracks for the revised elevated lines now under reconstruction. The moving took place Sunday, Aug. 22, 1915, on one tide. A description of the method of transferring the spans will appear in the next issue of *Engineering News*.

REPLACING HARLEM RIVER ELEVATED RAILWAY SWINGBRIDGE

Looking north, Sunday, Aug. 22, 1915. Old bridge at right, single-deck double-track rim-bearing bridge built 1878. New bridge at left, a double-deck four-track center-bearing bridge

Ordinance on Fire Safeguards

In continuation of the work on the revision of the Building Code of New York City, Rudolph P. Miller, expert to the building committee of the Board of Aldermen, has drafted an ordinance, "Safeguards Against the Spread of Fire," replacing the old section, "Fire Walls and Shafts."

Fire walls are required to be constructed like exterior walls. Openings are restricted to 80 sq.ft. in area and must be provided on each side of the wall with approved automatic fire doors.

Fire partitions (providing an area of refuge) must be 8 in. thick if brick or 4 in. if reinforced concrete. Shafts over 9 sq.ft. in area must be walled with fire partitions. If not over 9 sq.ft. they may be walled with 2-in. tile, concrete or the like, properly tied. Existing elevators in nonfireproof public buildings are required to be inclosed the same as new elevator shafts.

Fire doors (or shutters) or fire windows are required on openings above the first story in fireproof and nonfireproof business buildings which are more than 40 ft. in height, unless any such openings are more than 30 ft. from any other exterior opening or more than 50 ft. above the adjoining roof. Openings to fire escapes will practically have to be built of fire windows inasmuch as the fire shutters or fire doors that might be provided must be made self-closing and so arranged as not to obstruct. A

provision is made, suggested by the National Board of Fire Underwriters, that wherever in buildings over 40 ft. high no fire windows are called for, a 3-ft. space of solid masonry must be left between the top of one window and the sill of the window above it. It is also provided for that all fire doors and shutters shall be closed at night by the tenants, unless equipped with automatic devices.

Wire glass when permitted or required must be used in panes not greater than 720 sq.in. and must be at least ¼ in. thick.

American Port Statistics in Foreign Trade, 1914-15

The rank of the different domestic ports in handling the unprecedented foreign trade of the United States in the fiscal year ended June 30, 1915, is indicated by the following compilation prepared by the Bureau of Foreign and Domestic Commerce of the Department of Commerce. It shows that ten districts handled 85 per cent. of the total trade. New York alone is credited with 48 per cent., having increased its leadership during the year. In 1914, 45 per cent. passed through its custom house.

The following table shows the total trade of the 10 leading customs districts as indicated by the commerce passing through the custom houses at their leading ports, with comparisons covering the fiscal years 1914 and 1915:

	Total Foreign Trade		Imports		Exports	
	1914	1915	1914	1915	1914	1915
Ports and Districts	(In Millions of Dollars)					
New York	1905	2125	1040	931	865	1194
New Orleans	283	289	89	80	194	209
Boston	226	260	160	153	66	107
Galveston	268	240	12	10	256	230
Philadelphia	161	164	96	73	65	91
San Francisco	130	158	67	76	63	82
Baltimore	144	157	34	25	110	132
Detroit	129	143	26	25	103	118
Seattle-Tacoma	110	136	55	68	55	68
Buffalo	118	105	30	31	88	74
Total, 10 districts	3474	3777	1609	1472	1865	2305
Total, all other	785	666	285	202	500	464
Grand totals	4259	4443	1894	1674	2365	2769

Of 54 Water-Treatment Plants in New Jersey in the middle of the present year, 23 were filters for protection against water pollution, 22 were plants for iron removal, 1 was for the removal of carbon dioxide, and 8 were for disinfection purposes.

Bids for Reconstructing Compton Hill Reservoir, St. Louis, Mo., received on Aug. 13, 1915, ranged from $280,187 to $385,615; the estimated cost was $410,000. The lowest bid was submitted by the Hiram Lloyd Building & Construction Co. E. E. Wall is water commissioner.

Reinforced-Concrete Retaining Walls totaling 1½ mi. in length will be used in raising portions of the levees of the Kaw Valley Drainage District, Kansas City, Kan. (see "Engineering News" of Aug. 19, 1915, p. 382). Bids for these walls are to be received on Sept. 3. S. H. McNeil is chief engineer of the Kaw Valley Drainage Board.

United States Naval Radio Stations now number 47, of which 21 are open to commercial messages, while the others are reserved for official business only. A station at Point Isabel, Tex., and a Great Lakes training school station are under construction. Plans have been made for radio communication between San Francisco and Manila by way of Honolulu, but work on this project will not be commenced until fall.

The Topographic Survey of Ohio will be completed, according to present estimate, by the end of 1915. Only a small strip in the southwestern part of the state remains to be surveyed. The work is being carried on jointly by the United States Geological Survey and the State of Ohio. Prof. C. E. Sherman of the department of civil engineering of Ohio State University is inspector for the state in the work, which has been in progress since 1900. The scale of the map made from the survey is 1 in. to the mile.

Increased Mileage of Concrete Roads—According to a new bulletin of the United States Department of Agriculture, the estimated amount of concrete pavement in the United States in 1914 was 19,200,000 sq.yd.; in 1909 it was only 364,000 sq.yd. In addition to discussing the engineering details of construction, the bulletin calls attention to the fact that ordinarily from one-third to one-half of the total cost of constructing a concrete pavement is for labor after the materials are delivered. This emphasizes the importance of efficient organization and proper equipment. Failure to take these factors into consideration frequently results in adding from 10 to 20 per cent. to the cost of a concrete pavement.

The Production of Spelter in the United States for the first six months of 1915 amounted to 207,634 tons made from domestic ores and 8898 tons from foreign ores, a total of 216,534 tons, as compared with 177,991 tons for the preceding six months and 175,058 tons for the first six months of 1914, according to C. E. Siebenthal of the United States Geological Survey. In addition there was produced by distillation from drosses and skimmings 13,546 tons of secondary spelter. No statistics were obtained of the spelter produced by remelting skimmings, drosses, etc., but it was probably not less than 12,000 tons. The total output of spelter from both ore and skimmings was therefore about 242,000 tons, or at the rate of 484,000 tons per year. The number of retorts in operation in June, 1915, was about 127,000. Additional retorts to the number of over 32,000 have since been completed, are under construction, or are planned. The apparent domestic consumption for the first six months of 1915 was 160,906 tons.

The Illinois Road Law, known as the Tice Road Law, has been amended so that the county boards now have the power to determine the type of pavement to be constructed in their respective counties. This was formerly, and more properly the duty of the State Highway Department, which, with its experienced and technically trained staff, is better equipped to determine the most economical type of road surfacing and less liable to be influenced by agents for particular road-building materials. An amendment provides that full information as to the cost of different types of construction shall be published before the county board decides what material shall be used. Provision has been made in the law so that the county boards may purchase various kinds of high-priced machinery and lease it to the townships. It has been proved that few townships can afford to purchase their own machinery of this class (tractors, rollers, etc.). Road dragging by the townships is compulsory. From $3 to $5 per mile must be used for this purpose every year. The recent legislature appropriated $2,000,000, and as much more is to be raised by the counties for state-aid road work, so that $4,000,000 is available for highway work in Illinois during the next two years.

PERSONALS

Mr. J. H. Lendi, formerly assistant to Mr. A. G. Carlson, Mechanical Engineer of the Universal Portland Cement Co., Chicago, has been made Electrical Engineer in the engineering department of that company.

Mr. George F. Stickney, M. Am. Soc. C. E., former Consulting Engineer of the Lake Erie and Ohio River Canal Board, Pittsburgh, Penn., has opened an office in the Arkay Bldg., Albany, N. Y., for the general practice of civil enginering.

Dr. John W. M. Bunker has resigned from the department of hygiene and sanitation of Harvard University to organize and direct the bacteriological research department of the Scientific Laboratories of the Digestive Ferments Co., Detroit, Mich.

Mr. Charles Upham is Chief Engineer of the Coleman du Pont Road, Inc., with offices in Wilmington, Del. Work on the Coleman du Pont road has been resumed, and approximately 20 mi. of concrete pavement will be put under construction before Nov. 1.

Mr. Joseph A. Caviezal, Superintendent of the Alabama, Tennessee & Northern R.R., York, Ala., has been promoted to be General Superintendent, with headquarters at York. The office of Superintendent has been abolished, and Mr. Caviezal will be in charge of the operating and maintenance departments.

Mr. G. R. G. Conway, M. Can. Soc. C. E., whose resignation as Chief Engineer of the British Columbia Electric Ry. Co., Ltd., and the Vancouver Power Co. was noted in "Engineering News" of July 15, has opened an office at 409 Royal Bank Bldg., Toronto, for the general practice of civil and hydraulic engineering.

Mr. Howard B. Waha, who graduated in civil engineering from the Pennsylvania State College in 1909, and who has

been employed in engineering work wtih the United States Forest Service in New Mexico and Arizona since graduation, has accepted the position of Assistant Professor of forest engineering in the New York State College of Forestry, Syracuse University.

Mr. Kenneth A. Heron, formerly connected with the Modesto Irrigation District, has been appointed Chief Engineer of the Goose Lake Valley Irrigation Co., Lakeview, Ore. This company has an irrigation system under construction that will ultimately include a large part of the Goose Lake Valley in Oregon. About 15,000 acres will be brought under irrigation very soon.

Mr. H. C. Phillips, M. Am. Soc. C. E., whose resignation as Valuation Engineer of the Atchison, Topeka & Santa Fé Ry. was noted in "Engineering News" of Aug. 12, has been appointed Assistant General Secretary (and not Assistant Secretary, as noted previously) of the Presidents' Conference Committee on Federal Valuation, with offices in the Commercial Trust Bldg., Philadelphia, Penn.

Messrs. Henry A. Wise Wood and Elmer A. Sperry have been selected by the American Society of Aëronautic Engineers as members of the Civilian Advisory Board of the United States Navy. Mr. Wood is President of the society and was a member of the aërodynamics laboratory committee appointed by President Taft in 1912. Mr. Sperry is the inventor of the Sperry gyroscopic stabilizer for aëroplanes, which a year ago was awarded the first prize of $10,000 by the French Government.

Mr. Farley Gannett, Assoc. M. Am. Soc. C. E., Consulting Engineer, of Harrisburg, Penn., and recently Engineer of the Water Supply Commission of Pennsylvania, has become associated with the Harrisburg Water Laboratories, which has been organized for the chemical and bacteriological examination of water, sewage, trade wastes, etc. The organization consists of Mr. Lesley McCreath, Jr., member of the firm of Andrew L. McCreath & Son, Analytical and Consulting Chemists, of Harrisburg, who still retains his interest in that firm; Dr. George R. Moffitt, City Chemist and Bacteriologist of Harrisburg Hospital, and head of the Moffitt Bacteriological Laboratory.

Messrs. Howard E. Coffin and Andrew L. Riker have been designated by the Society of Automobile Engineers to serve on the Civilian Advisory Board of the United States Navy, at the request of Secretary Daniels. Mr. Coffin is a graduate of the University of Michigan, and has been connected with the automobile industry since 1902. At present he is Vice-President of the Hudson Motor Car Co., Detroit, Mich. Mr. Riker is well known for his work in the design of heavy trucks, particularly electric vehicles. Both are prominent in the Mechanical Branch of the Association of Licensed Automobile Manufacturers, and both are Past-Presidents of the Society of Automobile Engineers.

OBITUARY

A. V. Nesbitt died Aug. 5 at the home of his sister in Boerne, Tex., at the age of 55 years. He was born in England and came to the United States when a young man. For a number of years he was engaged in railway surveying throughout the Southwest. Later he went to Mexico as a railway locating engineer. Of late years he had been connected with the Pearson Engineering Corporation, for which he made many preliminary reports on various projects in Brazil, Spain and Mexico.

Mendes Cohen, whose death was noted in "Engineering News" of last week, was an honorary member of the Engineers' Club of Baltimore. The biographical sketch given should have been supplemented by mention of the service rendered the City of Baltimore by his disinterested and active efforts as Chairman of the Sewerage Commission of that city from 1903 to 1909 to introduce a modern system of sewerage. During these six years Mr. Cohen without compensation devoted his untiring energy to the interests of the city to which he was strongly attached, and the investigation was probably the most comprehensive one that had been made up to that time for any American city. For many years and until over 80 he was active in the management of the Maryland Historical Society, first as Corresponding Secretary and then as President. Mr. Cohen was a man of great depth of character, high ideals, and of broad and liberal views. In manner he was a gentleman of the old school, dignified but sympathetic to a marked degree, winning the esteem and affection of his large circle of friends.—Kenneth Allen, New York City.

ENGINEERING SOCIETIES

COMING MEETINGS

NEW ENGLAND WATER WORKS ASSOCIATION.
Sept. 7-9. Convention in New York City. Secy., Willard Kent, Narragansett Pier, R. I.

TRAVELING ENGINEERS' ASSOCIATION.
Sept. 7-10. Annual meeting in Chicago. Secy., W. O. Thompson, N. Y. C. R.R., East Buffalo, N. Y.

NORTHWESTERN ROAD CONGRESS
Oct. 4-7. At Cedar Rapids, Iowa. Secy., Jas. P. Keenan, Milwaukee, Wis.

AMERICAN SOCIETY OF MUNICIPAL IMPROVEMENTS.
Oct. 12-14. Annual convention at Dayton, Ohio. Secy., Charles Carroll Brown, 702 Wulsin Bldg., Indianapolis, Ind.

SAN FRANCISCO MEETINGS

INSTITUTE OF RADIO ENGINEERS.
Sept. 13-18. Secy., David Sarnoff, 71 Broadway, New York.

PAN-AMERICAN ROAD CONGRESS.
Sept. 13-18. Secy., E. L. Powers, Road Builders Association, 150 Nassau St., New York City.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
Sept. 16-18. Secy., Charles Warren Hunt, 220 West 57th St., New York City.

AMERICAN SOCIETY OF MECHANICAL ENGINEERS.
Sept. 16-18. Secy., Calvin W. Rice, 29 West 39th St., New York City.

AMERICAN INSTITUTE OF MINING ENGINEERS.
Sept. 16-18. Secy., Bradley Stoughton, 29 West 39th St., New York City.

AMERICAN INSTITUTE OF ELECTRICAL ENGINEERS.
Sept. 16-18. Secy., F. L. Hutchinson, 29 West 39th St., New York City.

AMERICAN SOCIETY OF HEATING AND VENTILATING ENGINEERS.
Sept. 16-18. Secy., J. J. Blackmore, 29 West 39th St., New York City.

AMERICAN ELECTROCHEMICAL SOCIETY.
Sept. 16-18. Secy., J. W. Richards, Lehigh University, South Bethlehem, Penn.

NATIONAL DRAINAGE CONGRESS.
Sept. 17-21,

INTERNATIONAL IRRIGATION CONGRESS.
Sept. 20. Secy., Arthur Hooker, Sacramento, Calif.

MINING AND METALLURGICAL SOCIETY OF AMERICA
Sept. 20. Secy., F. F. Sharpless, 52 Broadway, New York City.

PACIFIC COAST ASSOCIATION OF PORT AUTHORITIES.
Sept. 20 and 21. Secy., Leo V. Merle, Ferry Bldg., San Francisco, Calif.

AMERICAN MINING CONGRESS.
Sept. 20-22. Secy., J. F. Callbreath, Washington, D. C.

INTERNATIONAL ENGINEERING CONGRESS.
Sept. 20-25. W. A. Cattell, Secy., Comm. of Management, Foxcroft Bldg., San Francisco.

INTERNATIONAL ASSOCIATION OF BRIDGE AND STRUCTURAL IRON WORKERS.
Sept. 20-30. Secy., Harry Jones, American Central Life Bldg., Indianapolis, Ind.

AMERICAN SOCIETY OF AGRICULTURAL ENGINEERS.
Sept. 21 and 22. Secy., F. M. White, Madison, Wis.

AMERICAN SOCIETY OF REFRIGERATING ENGINEERS.
Sept. 23 and 24. Secy., W. H. Ross, 154 Nassau St., New York City.

AMERICAN MINE SAFETY ASSOCIATION.
Sept. 23 and 24. Secy., H. M. Wilson, Bureau of Mines, Pittsburgh, Penn.

AMERICAN ASSOCIATION OF REFRIGERATION.
Sept. 23 and 24. Secy., J. F. Nickerson, 431 South Dearborn St., Chicago, Ill.

ILLINOIS GAS ASSOCIATION.
Sept. 27. Adjourned meeting. Secy., H. H. Clark, 72 West Adams St., Chicago, Ill.

SAFETY CONFERENCE.
Sept. 27-30.

INTERNATIONAL GAS CONGRESS.
Sept. 27 to Oct. 1. Secy., George C. Ramsdell, 29 West 39th St., New York City.

AMERICAN GAS INSTITUTE.
Sept. 27 to Oct. 1. Secy., George C. Ramsdell, 29 West 39th St., New York City.

AMERICAN ELECTRIC RAILWAY ASSOCIATION.
Oct. 4-9. Secy., E. B. Burritt, 8 West 40th St., New York City.

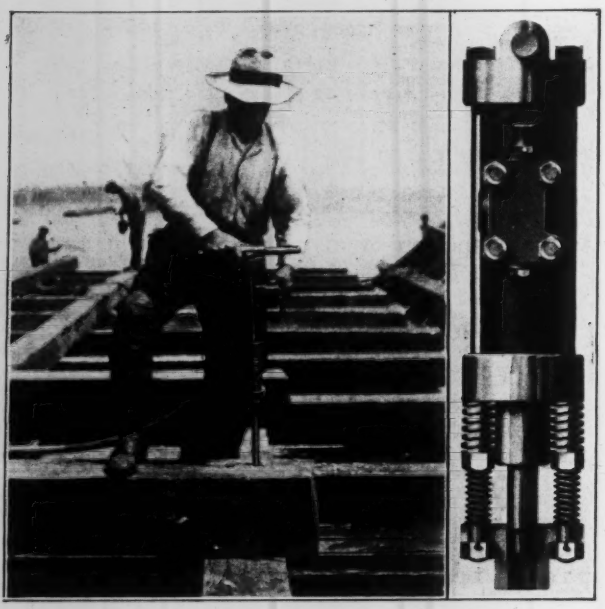
AMERICAN ELECTRIC RAILWAY MANUFACTURERS' ASSOCIATION.
Oct. 4-9. Secy., H. G. McConnaughy, 165 Broadway, New York City.

AMERICAN ELECTRIC RAILWAY ENGINEERING ASSOCIATION.
Oct. 4-9. Secy., E. B. Burritt, 8 West 40th St., New York City.

Appliances and Materials

Drill Attachment for Driving Spikes

The McKiernan-Terry Drill Co., New York City, has placed on the market a special attachment for its B-1 pneumatic-hammer drill, which facilitates the use of the drill for driving spikes in dock, crib or any heavy timber construction. The attachment, which holds the head of the spike, may be removed and the drill used for rock-drilling if desired. The following are the essential specifications: Length overall, 22 in.; cubic feet of air per minute at 80-lb. pressure, 70; size of hose, ½ in.; size of hexagon steel, 1 in.; weight complete, 55 lb.; cost, $85 net.



ATTACHMENT TO SMALL PNEUMATIC HAMMER DRILL FOR DRIVING SPIKES IN TIMBERS

* * *

A New Type of Diesel Oil Engine

Improvements on the Diesel oil engine, invented by Leonard B. Harris, of Philadelphia, have been embodied by the Southwark Foundry & Machine Co., of Philadelphia, in a new engine which is being built for marine and stationary service. The Harris engine is a two-stroke-cycle engine in which the scavenging pump generally used on European Diesel engines is replaced by a stepped piston, of which the upper and smaller diameter is the piston of the oil-engine cylinder and the lower and larger diameter acts as an air compressor and also as a cross-head and guide for the engine piston. These air-compressing pistons are likewise used for starting the engine, so that compressed air is not discharged into the oil-engine cylinder to cause its refrigeration just previous to the entry of the fuel. Another important feature of the Harris-Diesel engine is that there are no valves in the cylinder head, except the atomizer which introduces the fuel. The scavenging air is introduced near the bottom of the cylinder as the piston completes its stroke. By this construction the engine is able to start under load like a steam engine.

One of these engines has recently been installed on the yacht "Southwark," owned by C. P. Vauclain. This boat is 98 ft. long and of 16 ft. beam; the engine is of about 240 hp. During a trial trip on June 26 a run of 35 mi. was made at a speed of 12 mi. per hr. on about 29 gal. of Mexican oil.

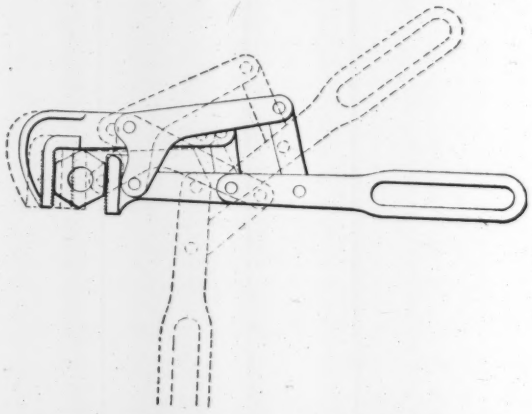
* * *

Automatic Control of Gas Lamps

Valves for the distant control of gas lamps are being made by the Lombard Governor Co., of Ashland, Mass. These are of a so-called "magnetic" type, with two electric circuits (two wires and a pipe return) turning on and off the gas and controlling the pilot flame. It was at first intended to use these for control of street lamps and a quite perfect clock was developed for use with them on lamp posts. The demand for valves for control of store and building lamps was greater than for street lighting, however.

New Pipe and Nut Wrench

A new design of quick-acting pipe and nut wrench, known as the "Hayward Automatic," and shown in the accompanying sketch, has been devised by the Hayward Wrench Co., 700 Cass Ave., St. Louis. Turning the wrench on the pipe or nut automatically adjusts its jaws and causes them to grip.



QUICK-ACTING PIPE AND NUT WRENCH

* * *

A Three-Wheeled Fire Fighter

A chemical engine suitable for small towns or large industrial works is being manufactured by the Davis Sewing Machine Co., Dayton, Ohio. It is equipped with a special Holloway type chemical tank of 35-gal. capacity, 200 ft. of F. D. chemical hose, extension ladder, fire extinguishers, ax, crowbar, pike pole, lanterns, etc. The motor is started by the rider in the saddle with a downward stroke on either pedal, and it is claimed that the machine can average from 30 to 45 mi. per hr. on fire runs. Its stream range is equal to the height of a four-story building, and its extension ladder equipment will reach the second story.

* * *

Pocket-Size Blasting Machine

A small blasting magneto, capable of firing three or four electric blasting caps, has been developed by the E. I. du Pont de Nemours Powder Co. for the convenience of coal miners, quarrymen, agricultural blasters, and for those generally who require small blasts. This little instrument is 4½ in. high by 3⅝ in. wide by 2½ in. thick, and weighs 4½ lb. It has rounded edges and is just the right size and shape for the side coat pocket. In operating the machine it is not necessary to find a level spot on which to stand it. It is held in the left hand, the right grasping the key. A sharp twist of the key generates the current and fires the shot. After use the key is removed and the cap is screwed on, providing a handle. The machine sells for $10 f.o.b., Pompton Lakes, N. J.

SMALL BLASTING MACHINE WITH FIRING KEY IN PLACE

When key is removed socket-cap shown in left-hand view can be screwed on keyhole, to serve as handle; or magneto may be carried in pocket